THE FIRE PERFORMANCE OF MOMENT-RESISTING NAILED CONNECTIONS BETWEEN HEAVY GLULAM TIMBER MEMBERS

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R. CHINNIAH

University of Canterbury
Christchurch, New Zealand.
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ABSTRACT

The fire performance of moment-resisting nailed connections between heavy glulam timber members is investigated. For protected steel and plywood gusseted joints, the time-temperature results achieved during a standard fire test (ISO 834) are simulated. The joints are loaded during this simulation to determine the load-deformation characteristics of the nails. Using these load-deformation relationships as input information to a computer model, the moment-rotation characteristics of these joints are computed. The application of the model to predict the fire performance of nailed joints, tested under full exposure is shown. Further application is indicated by analysing a knee joint of a portal frame. The temperature development along the length of the nails in these joints is also observed.

Nailed gusseted connections between glulam members have good fire performance when protected by gypsum plaster board. The nail temperatures in protected joints are too low to cause any serious undesirable effects on joint performance. The application of the computer model to predict the fire performance of nailed gusseted connections tested under full exposure has shown good correlation with test results. Calculations show that the deflections of the protected knee and apex joints of a typical portal frame are small and the frame as a whole has very good fire resistance.

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CHAPTER ONE

INTRODUCTION

1.1 GENERAL

Glulam timber, through its versatility and cost-effectiveness, is making timber construction more and more popular. Consequently there is a resurgence in the use of heavy glulam timber for various structures both in New Zealand (Hay, 1987) and worldwide.

The fire resistance and residual strength characteristics of heavy glulam timber are well established. However little is known about the fire performance of metal or timber connectors between glulam members. Connections are vital parts of the structural system, for their premature failure may seriously hamper evacuation of occupants and subsequent fire fighting. The overriding requirement in this respect which is specified in Finnish building regulation reads: "Metal components which constitute a load bearing element in timber structures shall be protected in such a way that they have at least the same fire resistance as the rest of the structure (Jonsson and Pettersson, 1985).

Overseas research indicates that the load capacity of exposed connections is reduced considerably during a fire. Steel connectors absorb heat and cause charring of adjacent timber, while timber or plywood gussets possess inadequate fire resistance. Further, the fasteners holding the connectors, themselves conduct heat into the wood, initiating softening, local charring and loss of anchorage (Hillis and Rozsa, 1978).

In the past decade, there has been a dramatic increase in the use of glulam timber in industrial buildings in New Zealand (Smith 1984). Current research might pave the way for use in multistorey construction as well. Many of these buildings incorporate moment-resisting connections, consisting of large nailed on steel or plywood gusset plates. Large nailed gusset plates are used much more in New Zealand than in other countries, partly because radiata pine timber can take many closely spaced nails without splitting. Lack of design information regarding the behaviour of these connections in a fire and the inapplicability of overseas guidelines are concerns for the local construction industry.

In recognition of the enormity of the situation, this project has been undertaken to investigate the fire performance of moment-resisting nailed connections between heavy glulam timber members. Both plywood and steel gussets with different protection materials in different thicknesses are investigated. Load-slip curves under standard fire conditions (ISO 834) are developed. This information is then input into a computer program to calculate the moment-rotation characteristics of the joint. The results of this investigation will be of great value to designers in deciding the type of protection to be used and its adequacy from a structural point of view.

The project described in this report is part of a larger project currently in progress at the Building Research Association of New Zealand. The BRANZ project has provided input information to this study, and has conducted full scale verification tests.

1.2 OUTLINE OF REPORT

This report consists of eleven chapters. Chapters two and three generally give some background information regarding the fire performance of timber joints and the protections used for these joints. In chapter four the fire performance of nails is discussed.

The experiments conducted as part of this study are discussed in chapter five, followed by the presentation of results in chapter six.

Chapter seven shows the load-slip relationships developed for nails in protected timber joints. Chapter eight outlines the computer model which uses the load-slip relationship of nails to determine the moment-rotation characteristics of the joint. The application of the computer model is indicated through a portal frame design example in chapter nine. Further application is given in chapter ten by using the model to predict the fire performance of timber joints tested under full exposure. Summary, conclusions and recommendations for future research form chapter eleven.

CHAPTER TWO

LITERATURE SURVEY OF FIRE PERFORMANCE OF JOINTS

2.1 INTRODUCTION

This chapter describes previous investigations that have been carried out to study the parameters affecting the fire performance of nailed joints. Most of the research in this field has been carried out in Europe, particularly in Germany and in Scandinavian countries. A preliminary literature survey was carried out by Olsen (1986) and the references cited by him provide up-to-date information on this subject. A survey on related subjects by Carling (1986) also provides useful information.

A number of investigations have been carried out to study the fire performance of nailed joints as well as the individual components. Joints of different geometry and of different gusset materials (plywood, steel, timber) have been studied. In some cases the joints were protected against direct fire exposure, while in others, they were left unprotected. The findings are briefly discussed in the following paragraphs.

2.2 DISCUSSION ON FIRE PERFORMANCE OF JOINT COMPONENT MATERIALS

The major factors influencing the fire performance of nailed joints in glulam timber can be grouped as below.

(1) Glulam timber properties

(2) Plywood gusset properties

(3) Steel gusset properties

(4) Nail properties

Each of these joint components is discussed briefly in the following paragraphs.

2.2.1 GLULAM TIMBER

The fire performance of heavy glulam timber is well established(Anon, 1976; Canadian Wood Council, 1977; Do and Springer, 1983; Hall et al, 1980; Kordina and Meyer-Ottens, 1977; Schaffer, 1984; Buchanan, 1987). During normal combustion of the exposed section, a charred layer is formed which continues to grow in thickness at a relatively slow but approximately constant rate. This charring rate varies depending on the size, type and density of the timber, permeability, moisture content, inherent defects and strain level, as well as the intensity of exposure and ventilation (Buchanan, 1987; Lie, 1972; Schaffer, 1984; Sauvage, 1985). The charring temperature is generally accepted as being around 288^{o}C to 300^{o}C (Hall et al, 1980; Malhotra, 1986). During exposure of wood to elevated temperatures, Fredlund (1979) has identified four characteristic zones parallel to the heated surface, as shown in figure 2.1.

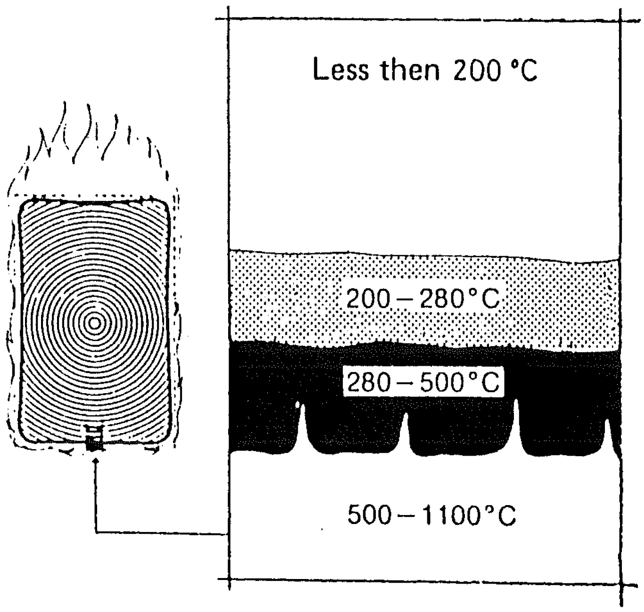
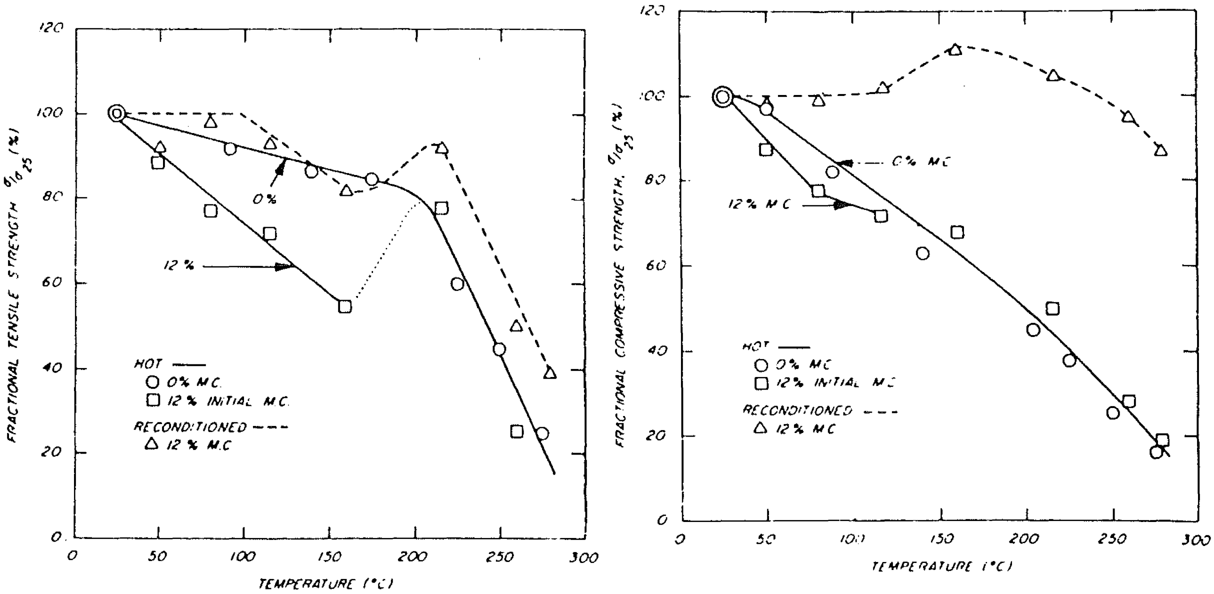


Figure 2.1 Characteristic zones in wood during pyrolysis.

The mechanical properties of timber, e.g., tensile, compressive and moduli generally degrade when exposed to higher temperatures (Springer and Do, 1983). The effects of elevated temperature on tension and compression strengths of wood is shown in figure 2.2 (Schaffer 1984).



A B

Figure 2.2 Tensile (A) and compressive (B) strengths as functions of temperature while hot as well as after cooling.

It can be seen that strength drops off quite rapidly with increasing temperature, although there is a discontinuity in the tension strength relationship at 12% initial moisture content. The dotted lines show that wood exposed to temperatures below about 200^{o}C regains almost all of the strength loss on cooling. This is useful when considering re-use of timber damaged by fire (Buchanan, 1987).

At temperatures less than 200^{o}C, the modulus of elasticity is relatively insensitive to temperature. A significant drop occurs over 200^{o}C, as shown in figure 2.3 (Schaffer 1984).

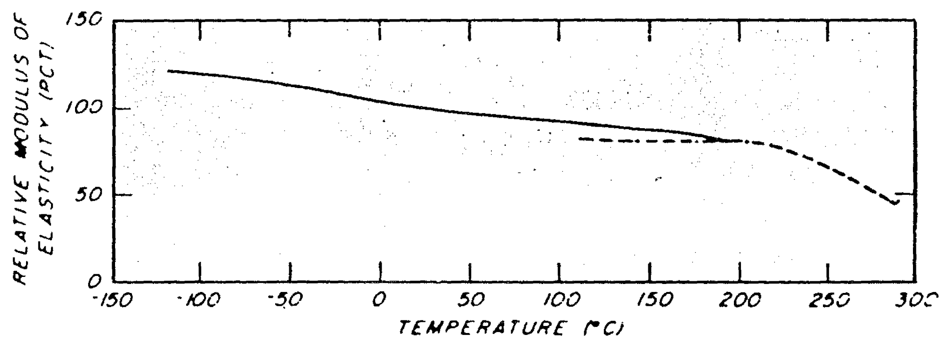


Figure 2.3 Effect of temperature on E parallel to grain at 12% moisture content. E is 100% at 20°C.

In terms of wood, from which the laminations are glued together, the following parameters can be expected to have some bearing on performance under fire conditions. These are,

(a) Timber species and density

(b) Moisture content

(c) Rate of charring

(d) Glue types

<u>(a) Timber species and density</u>

Different species of timber behave differently in a fire environment. The behaviour is largely dependent upon density and permeability of the timber. The less dense and more permeable species char at a higher rate. White Oak is especially resistant to charring due to its low permeability (Schaffer 1966, 1967; Tenning 1967; Rogowski 1967). Tests conducted by Malhotra and Rogowski (1967) also indicate the varying fire performance of different species. They tested glulam columns made of four different species (Douglas fir, European redwood, western hemlock and western red cedar). The columns were loaded and they point out that the differences in the structural strength are not responsible for the fire behaviour of the species as they were taken into account when computing the design loads. However, the densities of the four species being different, they conclude that these may have had some bearing on the performance by influencing the charring rate.

Swane and Vagholkar (1968) conducted tests to determine the effect of heat on the static withdrawal resistance of nails. Three species were included in their study (Radiata pine, messmate stringybark and blackbutt). They found that the density of the particular species is one of the factors influencing the dispersion of heat energy along the nail.

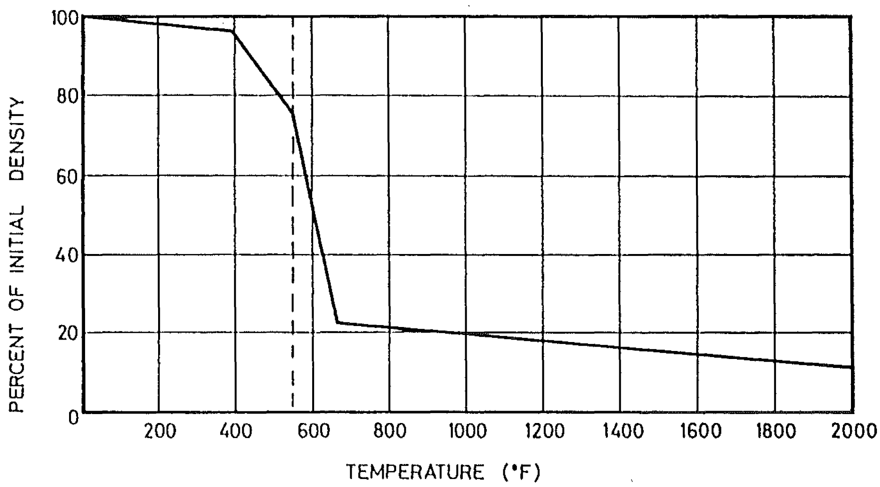


Figure 2.4 Function relating density and temperature

The density of wood decreases when exposed to fire, irrespective of the species involved. Thermogravimetric analysis by Tang (1967) has shown that the density of wood heated at a rate similar to that found in burning wood members decreases in the manner shown in figure 2.4.

(b) Moisture Content

Moisture content can have a large influence on the properties of wood at elevated temperatures in addition to affecting the burning rate (Knudson and Schniewind, 1975). MacLean (1941) noted that when heat is applied to one surface of a wood specimen, the moisture content on the heated side is reduced whereas the moisture content on the cool surface is increased. This can be accounted for by concluding that even though moisture is being driven out from the heated surface, a similar amount is being driven further into the wood (Hann, 1960). Hence, even though the average moisture content of the heated specimen is being reduced, moisture concentrations equal to or greater than the original moisture content do exist within the specimen.

Since moisture can be driven by a thermal gradient, the moisture distribution should be ascertained with respect to time. Schaffer and Duff (1965) examined the distribution in charring thick wood slabs of Douglas-fir and found the response with time. This is shown in figure 2.5. They observed the maximum moisture peak coinciding with a temperature level of 60^{o}C. There was only a 2% difference from the initial level of 16% well into the exposure period. They describe the moisture content peak as a "front" that moves into a fire-exposed section depending on the location of the interface between charred and uncharred wood (figure 2.6).

The 2% difference in moisture content and gradient shape was also measured by Dorn and Egner (1961) in a beam after fire exposure. In this case, the temperature level associated with the peak was about 71^{o}C. From this information it can be concluded that wood progressively becomes drier above 60 to 70^{o}C in fire exposure.

A change in moisture conditions during loading increases the shrinkage of the timber which can cause large deformations and cracking as shown by Armstrong and Kingston (1960). Perkitny (1965) claims that changes in moisture content as small as 1% exert an appreciable effect on creep.

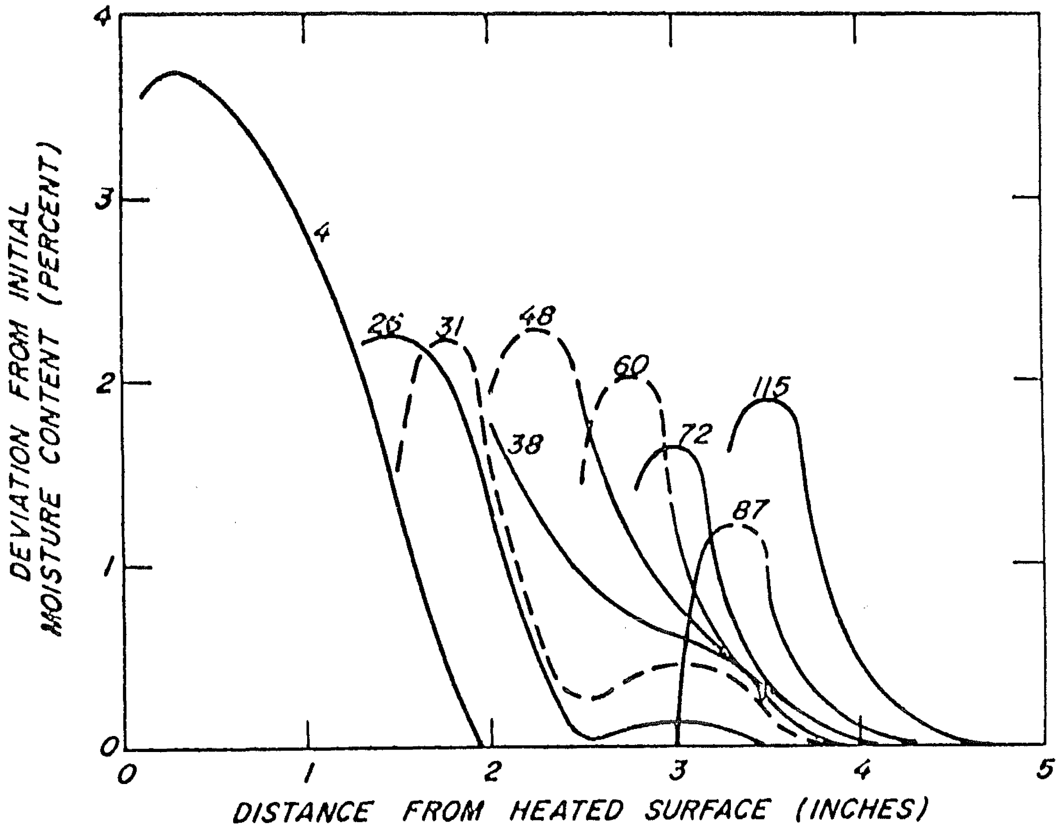


Figure 2.5 Moisture gradients in charring Douglas-fir slab (t = 190 mm) (time in minutes is indicated at each peak)

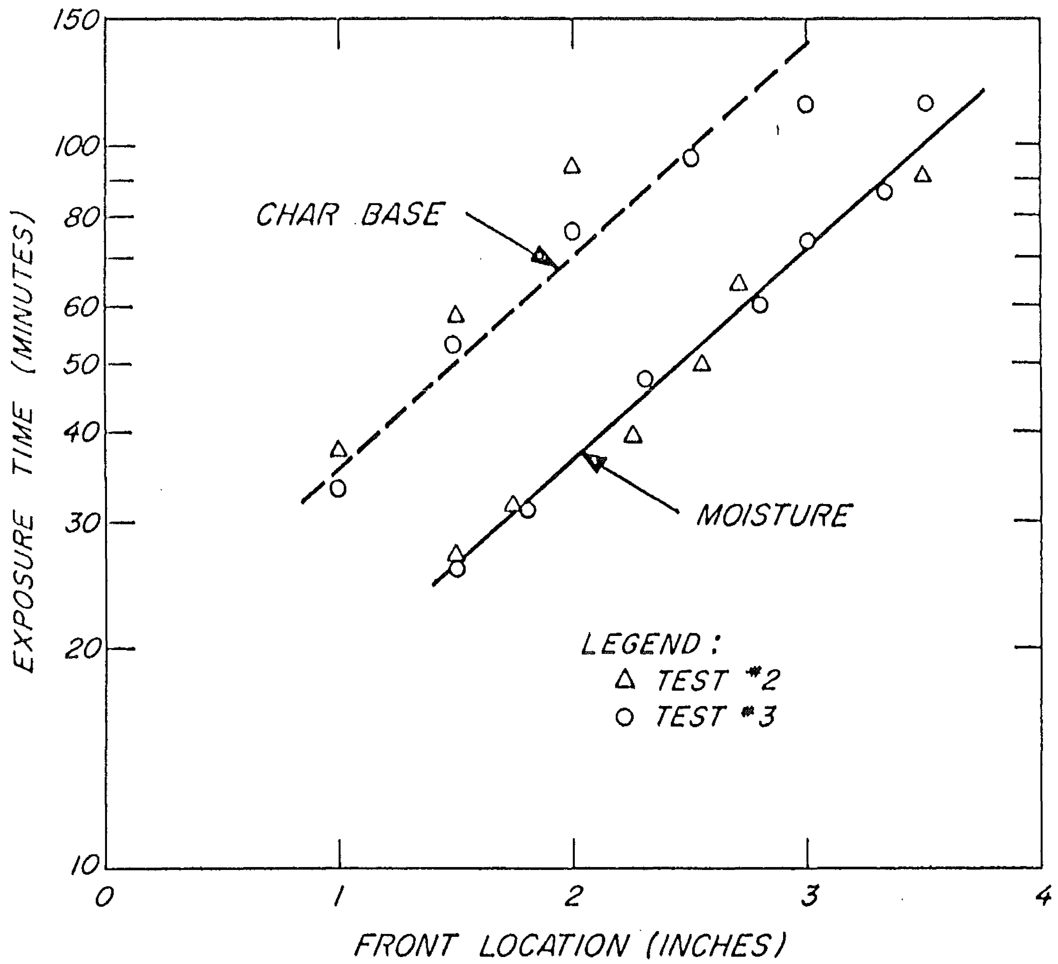


Figure 2.6 Location of moisture peak and char base in a charring 190 mm Douglas-fir slab.

(c) Rate of charring

This is the primary property of timber in determining the fire resistance. Charring is the process by which timber is heated, dried, pyrolised and converted into charcoal. These processes are all going on in different zones (Fredlund, 1979) in timber exposed to a fire. As the timber gets heated, its strength increases due to drying and then decreases as chemical degradation sets in.

The low thermal conductivity of wood and the outflow of decomposition products render the progression of heat into wood slow and predictable, under standard fire test conditions. The charring rate tends to be higher initially and then becomes constant with time. A higher initial rate is compensated for, under fire test conditions, by the ignition lag, which can be as much as 5 minutes for that timber exposed from the start of a fire test. For practical purposes the rate can be assumed to be constant from 15 to 90 minutes.

Many factors have been shown to influence the rate of charring (Sauvage, 1985). These include:

species of timber,

size of member (ratio of surface to volume),

moisture content,

temperature and heat flux,

duration of exposure to heat,

configuration of member,

timber defects,

degree of exposure (one or more directions)

ventilation

Of all the above, the species and the size of the member deserve special consideration. The effect of timber species is primarily density dependent although individual timbers do show characteristic charring rates outside this general relationship. The effect of size of members is not yet well defined but small members, especially, when heated from three or more sides either char more rapidly or show greater high temperature weaknesses beneath the charred zone. Sauvage (1985) indicates the following charring rates depending on the density, at 18% moisture content.

density > 600 kg/m3 .... 0.50 mm/min.

density = 400 kg/m3 .... 0.67 mm/min.

density = 350 to 400 kg/m3 .... 0.83 mm/min.

He further states that these figures apply to fire exposure approximating standard time-temperature conditions. A fire exposure period of at least 10 minutes and a minimum uncharred timber section of 30 mm after exposure is also specified.

The New Zealand design code, MP9 (SANZ, 1987) specifies a charring rate of 0.6 mm/min. for all species.

(d) Glue types

The type of adhesive employed in the bonding of large members can have a significant effect on charring, due to heat-induced delamination at a glue line that is uniformly heated such as the glueline on the bottom laminate on beams and outer laminates on columns. Generally, phenolic and resorcinol adhesives have established reliability under fire exposure because of their thermal and moisture stability. Neither influences charring rate (Dorn and Egner 1961; Imaizumi 1962; Rogowski 1967; Schaffer 1967, 1968). Casein-glued laminations appear to have charring rates comparable to phenolic and resorcinol-bonded wood if outer laminates are thick enough to meet the desired fire endurance time. For example, for one hour fire endurance the outer laminates must at least be 40 mm thick (Rogowski 1967; Schaffer 1968). Urea glue leads to both increased charring rate and separation of laminations as a heated zone develops in the timber. Most New Zealand glulam manufacturers use phenolic or resorcinol adhesives.

2.2.2 PLYWOOD

Very little information is available on the mechanical and thermal properties and charring rate of plywood at elevated temperatures. Ashton (1965) considered that the charring rate of plywood may be greater than that of solid timber, depending on the type of veneer and adhesive. He suggested that premature delamination may result if unsuitable glues are used, giving a higher rate of destruction. Meyer-Ottens (1967) reports that the charring through of plywood panels (DIN 68-705) varies with the square of the thickness (figure 2.7) and gives 44 minutes as the time for complete destruction of a 25 mm thick plywood panel. King and Yiu (1988) observed rapid charring and consequent deterioration of plywood, when it was used as a protection material for timber joints.

2.2.3 STEEL

Mild steel gussets are widely adopted for moment-resisting connections in glulam timber. Under fire conditions they may absorb large amounts of heat and cause charring of timber beneath them. The charring may be pronounced if the joint is loaded.

The strength and modulus of elasticity of steel decrease with an increase in temperature, with 550^{o}C being the critical value (Carling, 1986). The creep rate of steel increases at temperatures above 400^{o}C. Since a well defined yield point does not exist at elevated temperatures,

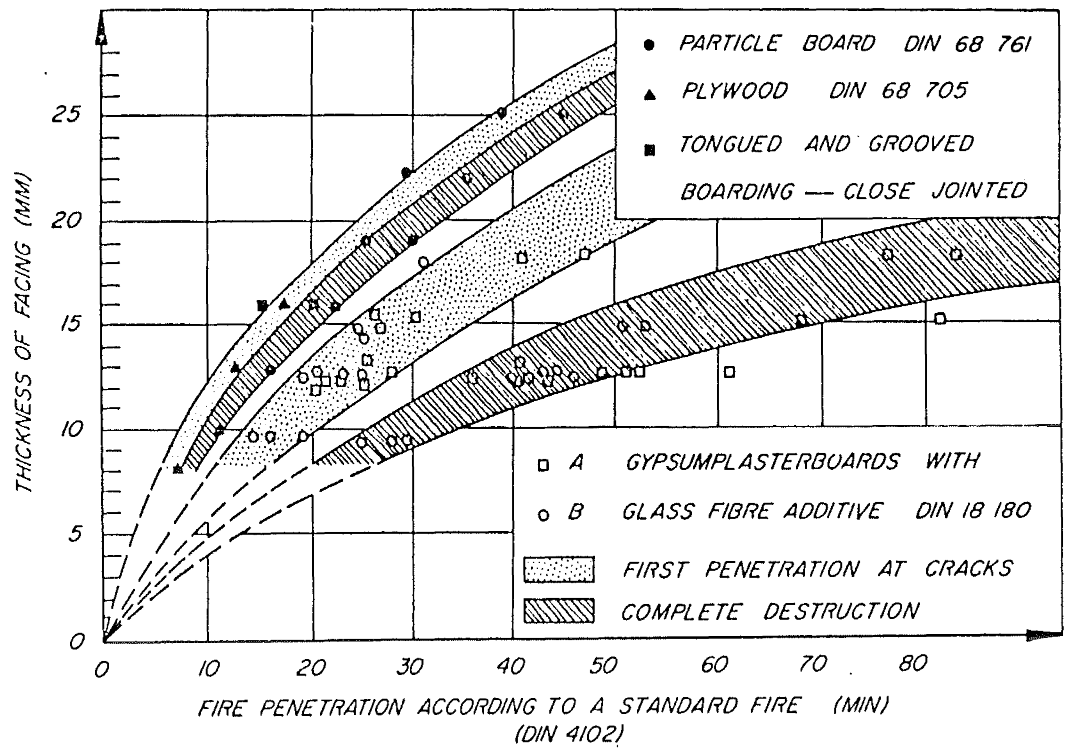


Figure 2.7 Fire penetration of plywood panels according to a standard fire (DIN 4102).

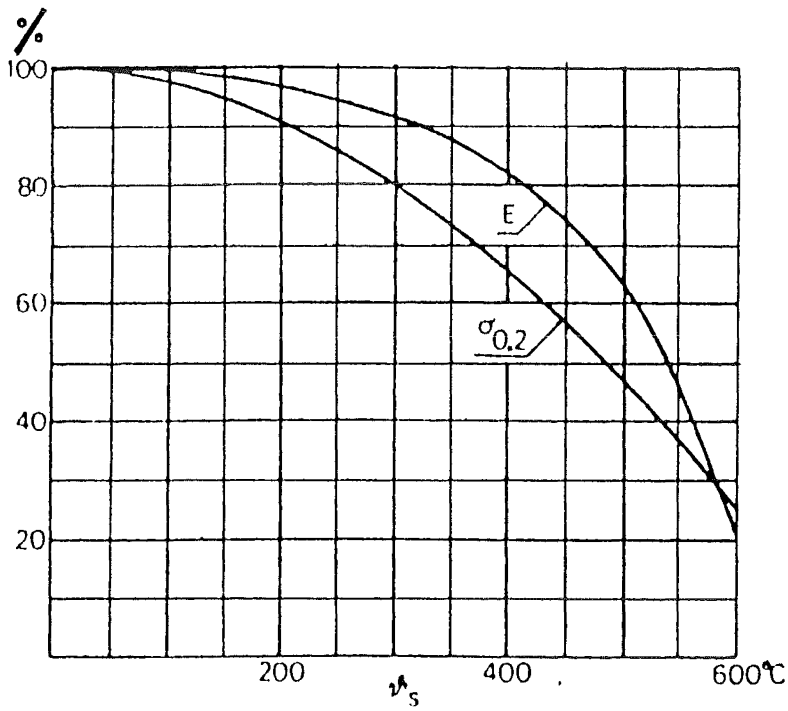


Figure 2.8 0.2%-limit and E as a function of the steel temperature.

the determination of the load capacity of joining details in mild steel is based on a 0.2% limit, i.e. the stress which gives a residual strain of 0.2%. The drop in modulus of elasticity and the stress corresponding to 0.2% strain with the steel temperature is indicated in figure 2.8 (Magnusson et al, 1974; Pettersson and Odeen, 1978).

2.2.4 NAILS

Nail properties at elevated temperatures can have an important bearing on the performance of the joint. The main factors that are to be considered are,

(a) Charring characteristics of timber around hot nails

(b) Temperature development along the length of the nail

(c) The effect of load on heated nails

(d) Embedding characteristics of heated nails

(e) Withdrawal characteristics of hot nails

There have been few investigations to study the fire performance of nails. Aarnio (1979) and Aarnio and Kallioniemi (1983) have investigated the charring characteristics of timber around unloaded nails. They included the effect of nail lengths and nail spacing in their study.

Leicester et al (1979) have studied the fire performance of different fasteners including nails. Of all the fasteners considered (nails, bolts, toothed plates and split ring connector) nails had a higher fire endurance. This they attribute to the geometry and embedment characteristics of nails.

Anderberg (1980) and Swane and Vagholkar (1968) have studied the effect of heat on the static withdrawl resistance of nails. These results are described in section 4.5.

It is also worth noting that the behaviour of nails is to some extent dependent on the type of gusset used. Nail heads in loaded plywood gusset joints can pull into the surface of the plywood due to hot conditions at the nail head whereas this type of failure is prevented in steel gusset joints. On the other hand nails may heat up more rapidly with steel gusset plates.

2.3 OTHER PARAMETERS INFLUENCING FIRE PERFORMANCE OF NAILED JOINTS

Apart from the joint component materials discussed above, there are other factors that are to be taken into account when studying the fire performance of nailed joints. These are,

(1) Joint considerations

(2) Loading considerations

(3) Fire characteristics

These aspects are discussed briefly in the following paragraphs.

2.3.1 JOINT CONSIDERATIONS

Joint characteristics play an important role in the fire performance of the structure. Careful consideration must be given to the size and the type of the gussets used, the fasteners used and the manner in which these are provided. The size of the member tends to determine the head and point side penetration of the nail. The type of gusset, be it plywood, steel or solid timber, determines the member's resistance to crushing by the shank of the nail.

Charring of timber under gusset plates will be much greater with steel gussets than with plywood gussets because steel is a very good conductor of heat. With both steel and plywood gussets, heat is conducted through the nails into the timber, causing softening and loss of anchorage of nails (Hillis and Rozsa, 1978).

Aarnio (1979) and Aarnio and Kallioniemi (1983) suggested that although the use of plywood or timber gussets may result in a larger undamaged timber section than connections involving steel gussets, the fire resistance of the joint is similar in both cases. Also the high temperatures at the nail head may char the adjacent gusset material, thus initiating a premature failure.

Further, the direction of loading relative to the grain of the member and the gap between members at the joint interface may also influence the fire performance of the joint.

2.3.2 LOADING CONSIDERATIONS

The type of loading and the load level play a dominant role in the fire performance of the joint. This has been borne out by the tests conducted by Leicester et al (1979). Their double lap spliced nailed joints were loaded in tension and they comment that these joints performed well because the deformation of the nails resulted in the faces of the timber being pressed tightly together, thus protecting the highly stressed portions of the wood and the nails.

One particular difficulty in assessing the results of the loaded joints, produced by various researchers is, their lack of correlation with the dead, live, wind or snow loads specified in the particular standard. Leicester et al (1979) have applied a mixture of dead and live loads which were roughly 0.36 times the average short term ultimate tensile strength of the connection when loaded without a fire environment. This is a higher load than would normally be expected during a fire.

At present MP9 (SANZ, 1987) specifies that structural members in fire should be capable of withstanding the design loading of 2/3 of wind load based on a 5 year return period wind gust velocity, together with 75% or 50% of the live load for storage occupancies and other areas respectively. This applies to the connections also if they are expected to have compatible structural performance. It also specifies higher than normal allowable stresses to be used for calculations under fire conditions. Overseas fire design codes specify lower loads to be considered in conjunction with fire.

2.3.3 FIRE CHARACTERISTICS

The fire behaviour of a structure very much depends on the type of thermal exposure. Fire tests of connections are normally performed in accordance with time temperature conditions in recognised standards, e.g., ISO 834 (1975), AS1530:PART 4 (1985), BS 476:PART 23 (1987), ASTM E119 (1988) or the standard of the particular country.

In cases of severe local exposure, the joint may experience intense damage. Load bearing and separating constructions of combustible materials may themselves provide considerable additional input in the event of fire. These aspects are not taken into consideration in furnace tests according to ISO 834, in which the input power of the furnace is regulated in such a way that the prescribed curve of the furnace temperature T as a function of time t conforms to the following equation

$T - T_0 = 345 \log_{10} (8t + 1)$ where,

t = time in minutes

T = furnace temperature at time t (^oC)

T_0 = furnace temperature at time t equals 0 (^oC)

In a given furnace, this requires an input power which is considerably less in a test on a structure of combustible material than in a test on a non-combustible material, particularly if the latter absorbs a large amount of the input power. However in a real fire, most combustible fuel is in the contents and the contribution of structure to fuel may be negligible.

A more realistic way of viewing thermal exposure is to describe it in terms of a time curve relating either to the gas temperature in the fire compartment, or to the heat flow rate towards the load bearing structure. In the individual case, the actual curve can be determined on the basis of the heat and mass balance equations of the fire compartment, or from available manuals. Figure 2.9 gives an example of such data, taken from Liber Förlag, Stockholm, 1976.

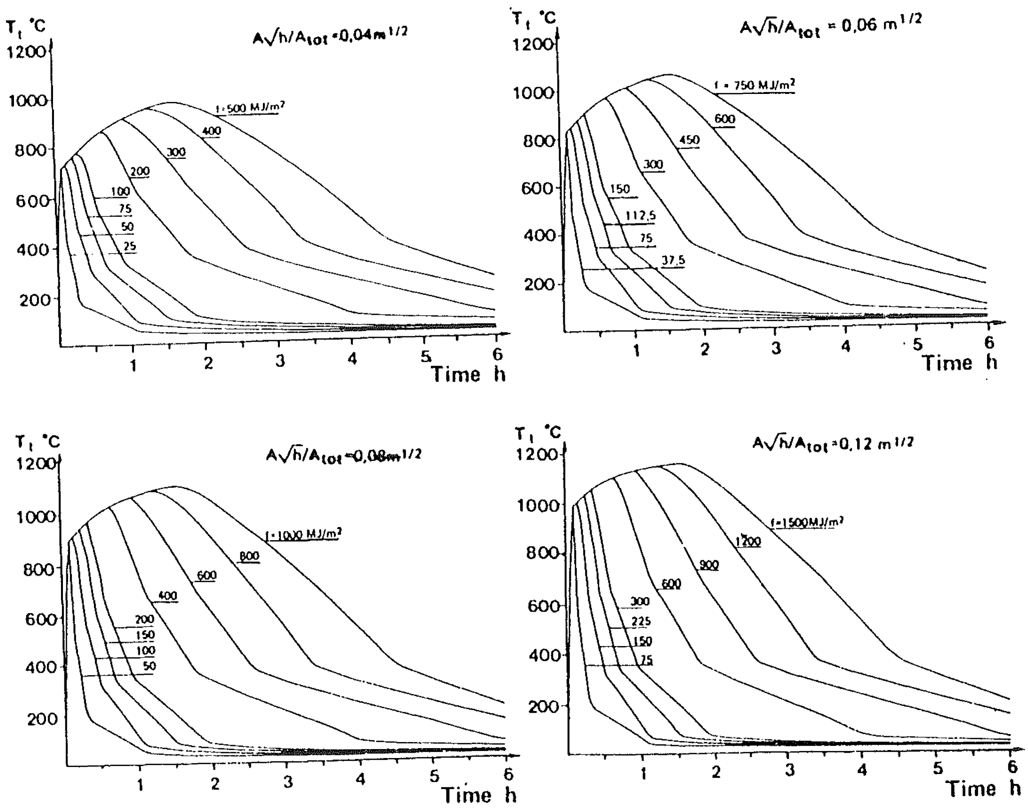


Figure 2.9 Examples of gas temperature time curves for a real fire as a function of fire load and opening factor.

Leicester et al (1979) have attempted a less severe fire condition, i.e., a typical fire in a residential building, as an alternative to the standard fire condition and observed improvement in the fire performance of the nail joints tested. Baldwin (1974) has pointed out that in assessing the fire risk, only 10% of all fire ignitions reach the point of flashover and many fires are brought under control in a shorter time than the code specified fire resistance period.

2.4 FIRE PERFORMANCE OF UNPROTECTED NAILED CONNECTIONS

Currently there are no analytical methods available to determine the behaviour and load bearing capacities of protected or unprotected nailed connections. Gibson (1984) has performed calculations on unprotected moment-resisting nailed plates based on the time-temperature characteristics for a typical industrial building fire. Based on an assessment of the buckling characteristics of steel plates at elevated temperatures (ECCS, 1974), he indicated a fire resistance period of about 12 minutes.

Various types of unprotected connections have been investigated experimentally. Although none of them resemble the form nor geometry of the moment-resisting connection that is being investigated in this study they do provide useful insight into certain aspects. A summary of these findings is given in tables 2.1 and 2.2 reproduced from King and Yiu (1988).

In the case of unprotected nailed plywood gusset connections, both Aarnio and Jackman observed that the high temperature at the nail head caused the surrounding timber to char, offering very little resistance against being pulled through eventhough the nail shank is still fixed adequately due to low temperatures at the nail tip.

Kordina and Meyer-Ottens (1983) pointed out the risk of instability

TABLE 2.1: UNPROTECTED NAILED STEEL GUSSET CONNECTIONS

Reference	Failure time (minutes)	Failure criteria and loading	Note
Lihavainen (1976)	7	insulation (unloaded)	assumed failed when timber surface temperature = 300°C
Silcock *et al* (1977)	7 to 15	collapse (loaded)	timber roof truses with nailed plate connectors
Baldwin and Ransom (1978)	10 to 15	not known (loaded)	metal surface fastenings in trussed rafter roof
Rimstad (1979) and Bakke (1978)	14	insulation (unloaded)	timber surface temperature = 300°C
Ahlen and Mansson (1979)	16 to 24	deformation (shear load)	fitted sleeve joint

TABLE 2.2: UNPROTECTED NAILED PLYWOOD GUSSET CONNECTIONS

Reference	Failure time (minutes)	Failure criteria and loading	Note
Hviid and Olesen (1977)	12	collapse (loaded)	gusset, fire retardant impregnated
Leicester *et al* (1979)	33	deformation (loaded)	failure, when the joint extended 10 mm
Aarnio (1979)	17.5	collapse (loaded)	nail heads pulled through the gusset
Jackman (1981)	-	collapse (loaded)	the plywood failed when its thickness was reduced by charring to a point where the stress in the gusset was close to the ultimate strength of the cold material
Kordina and Meyer-Ottens (1983)	22	-	gusset of swan timber or glulam timber
TRADA (1985)	30	deformation (loaded)	joint expected to sustain the design load until the end of the test

of timber gussets in fire and proposed minimum thicknesses for gussets related to the dimensions of the connected members as well as the fire resistance required. Ahlen and Mansson (1979) found, that the rate of charring increases in proportion to the load when transferred to the timber by steel.

These studies indicate that unprotected loaded gusset connections cannot achieve one hour fire resistance irrespective of the failure criteria. Fire affects both the strength and deformation characteristics of the connection. Further, any permanent degradation or damage to the component materials is also undesirable. Thus, it becomes necessary to protect the connections.

2.5 FIRE PERFORMANCE OF PROTECTED CONNECTIONS

The fire performance of both protected steel and plywood connections have been studied and a summary of these studies is presented in table 2.3, reproduced from King and Yiu (1988).

2.6 CODE RECOMMENDATIONS

For single storey industrial buildings built near property boundries, NZS 1900 Chapter 5 indirectly specifies the fire performance requirement of moment-resisting connections by requiring that the structure supporting the external wall possess a fire resistance of one hour. This implies that the connections in this type of construction should also achieve the same fire resistance.

MP9 (SANZ, 1987) contains the following recommendations for joint details.

(1) The charring rate of 0.6 mm per minute shall apply to exposed timber surfaces and timber surfaces in contact with or adjacent to unpro-

TABLE 2.3: FIRE PERFORMANCE OF PROTECTED DETAILS.

Reference	Protection	Failure time (minutes)	Note
STEEL PLATES[1]:			
Lihavainen (1976)	30 mm thick mineral wool	37-43	-
Rimstad (1979) and Bakke (1978)	3 layers of 'Unitherm' fire resistant coating	28	8 and 16 mm thick steel plates: coatings have no effect until beyond 150 to 200°C and the temperature delay is about 10 minutes at 300° to 500°C
	20 mm thick fire resistant plaster	33	the plaster is effective at an early stage of the fire giving a temperature delay of about 15 minutes
Aarnio (1979) and Kallioniemi (1983)[2]	mineral wool: 30 mm thick 50 mm thick	35 65	assumed failure is when the timber surface temperature reaches 300°C
	5.5 mm thick asbestos-cement board placed between timber and metal plate	-	charred depth reduced by 30% compared with uninsulated plate after 20 minutes of testing
	glulam timber boxes: 40 mm thick	-	temperature at gusset: increased by 15°C after 30 minutes
	70 mm thick	-	no increase at all
	particle board boxes filled with mineral wool	-	temperature in steel less than the 300°C after 30 minutes, the particle board is completely charred but mineral wool is still in place
PLYWOOD GUSSETS:			
Hviid and Olesen (1977)	plywood	37	plywood is fire retardant impregnated; failure by joint rupture under loaded conditions
Kordina and Meyer-Ottens (1983)	sawn timber or glulam timber 24 mm thick	48	timber protection for gusset only; loaded joint failure by loss of load bearing capacity
Kordina and Meyer-Ottens (1983)	40 mm thick	68	timber protection for gusset only; loaded joint failure by loss of load bearing capacity

Note: [1] All the steel plates are unloaded.

[2] Both nailed, glued and nailed boxes as well as boxes provided with a 1 mm gap at the joints, were tested and the fire performance was found to be more or less uniform.

tected metal details.

(2) Timber members glued with thermosetting glue may be considered as 'solid' timber for fire resistance design. However members nailed or bolted together shall not be treated as one section.

(3) Cracks, gaps and concealed spaces on the timber surface are likely to have a flue action during fire and should be avoided.

(4) Metal fasteners exposed to fire shall be embedded into the residual section with the countersink holes plugged and/or covered with suitable protecting materials, e.g., timber, plasterboard, asbestos insulation board or equivalent.

Attention is also drawn to proper fixing of the protection materials. However no information is provided regarding the thickness and arrangement of the protections.

Overseas standards specify various criteria for the design or failure of unprotected connections or the protection requirements. These are summarised in table 2.4 reproduced from King and Yiu (1988).

Table 2.4: OVERSEAS STANDARDS

Reference	Protection	Note
NS 3478(1981) Norwegian Standards Institute	-	materials which are in direct contact with timber must be properly insulated such that their temperature does not exceed 300°C during the fire.
Ministry of Internal Affairs,Finland Building Regulation, Part 5 (1977)	timber,suitable boards or mineral wool boards	Ditto and load bearing metal components should be insulated.
British Standards Institution(1978) BS 5268:Part 4	timber board Asbestos-cement board or equivalent	metal parts should be buried deeply into the timber such that they lie within residual section or be covered with suitable material
German Standards Institution DIN 4102:Part 4 (1981)	timber 24mm and 40mm thick respectively	corresponds to fire resistance of F30-B and F60-B respectively
DS 413 (1982)	wooden plug or other types of insulation	assumes that the heat transmitting metal parts are protected against heat damage
State Planning Commission PFS 1984:1 (1984)	-	assumes decrease in loadbearing capacities of steel components at elevated temperatures.No load transfer between components and charred timber and the characteristic loadbearing capacity is increased by 50% in a fire compared with the normal situation;asbestos cement boards are no longer allowed for fire protection

CHAPTER THREE

FIRE PROTECTION OF JOINTS

3.1 INTRODUCTION

Research done so far clearly indicates the poor fire performance of exposed connection details. Thus they form a possible weak link in an otherwise strong structural system (heavy glulam timber construction). Therefore, protection of these connection details becomes mandatory if the glulam is required to be fire rated. The principal aim is to reduce exposure of the timber to fire by insulating as well as reducing radiation and oxygen supply (Barnett, 1984). This chapter outlines the types and forms of protection used and their fire performance. The effects of gaps and openings in their detailing is also discussed.

3.2 TYPES AND FORMS OF PROTECTIONS

There are several ways of protecting a joint from direct fire exposure. Some of the commonly used methods are indicated below.

(1) Embedding the fasteners (nails, bolts etc.) into the timber construction and plugging the holes with wooden plugs.

(2) Encasing the joint using timber components.

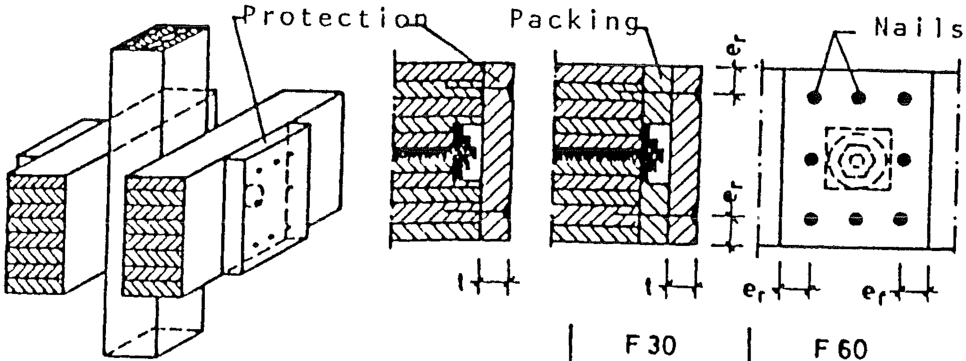
(3) Use of fire resistant plasters or board materials such as gypsum plaster, mineral wool, gypsum-vermiculite, gypsum-perlite or sprayed mineral wool.

(4) Application of fire resistant intumescent coatings.

The fire performance of these protections have been studied and the findings are described below.

3.2.1 EMBEDDING

Embedding is appropriate where fasteners such as bolts and glued-in screws connect two members. Aarnio and Kallioniemi (1983) studied the temperature development during a standard fire in a bolt which was embedded into the side of the timber with a 40 mm thick wooden plug glued into the hole. The temperature increase in the bolt after 20 minutes of fire was insignificant. The wooden plug had charred in the same way as the timber beam as a whole. The protections with timber coverings of thicknesses 20 mm and 40 mm are classified as class F 30-B and F 60-B respectively according to DIN 4102, as indicated in figure 3.1 (i.e. 30 minutes and 60 minutes fire resistance respectively).



	F 30	F 60
e_r [mm]	30	60
t (mm)	20	40

Figure 3.1 The required thicknesses of protection for bolted joints.

For a column base as shown in figure 3.2, Kordina and Meyer-Ottens (1983) estimate that class F30 can be achieved by applying fire resistant coatings to the flanges of the steel section and protecting the bolts by embedding. The covering thickness for the bolts in the form of glue in timber plugs should at least be 20 mm. For class F60 they recommend coatings of mineral wool or fire resistant plaster to the steel section and a covering thickness of 40 mm for the bolt.

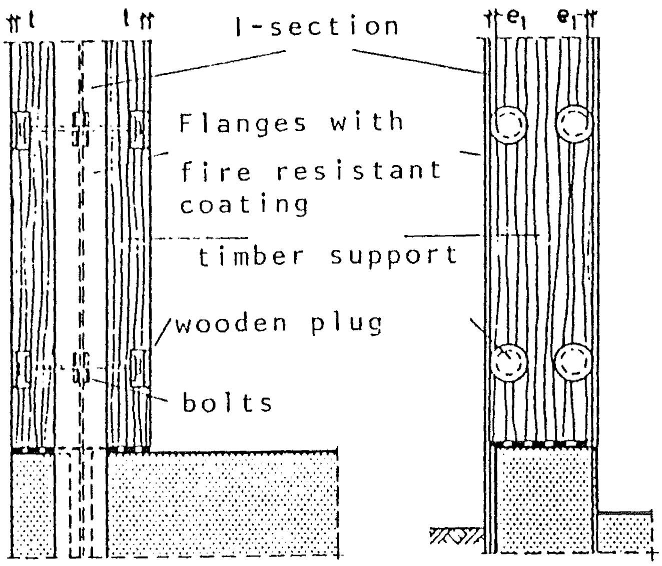


Figure 3.2 Fixed column in fire class F30.

Glulam timber columns with glued-in screws as shown in figure 3.3 have been tested under 60 minutes of fire by Aarnio and Kallioniemi (1983). The temperatures in the screw and at the interface between the beam and the column were recorded. The results are given in figure 3.4. The joint was not loaded. Aarnio and Kallioniemi found that the beam and the column, from fire resistance point of view, have functioned as a

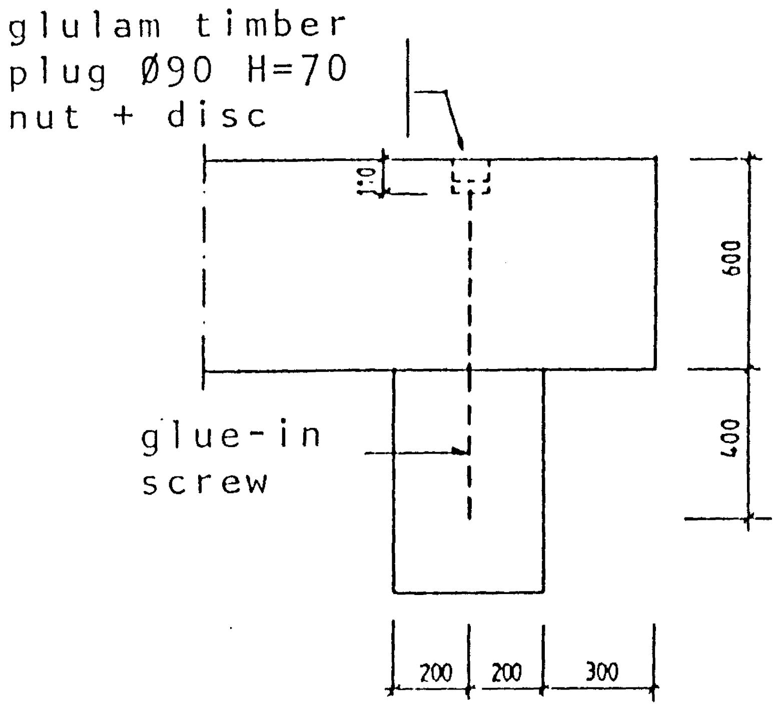


Figure 3.3 Fire tested beam-to-column joint with glue-in screw.

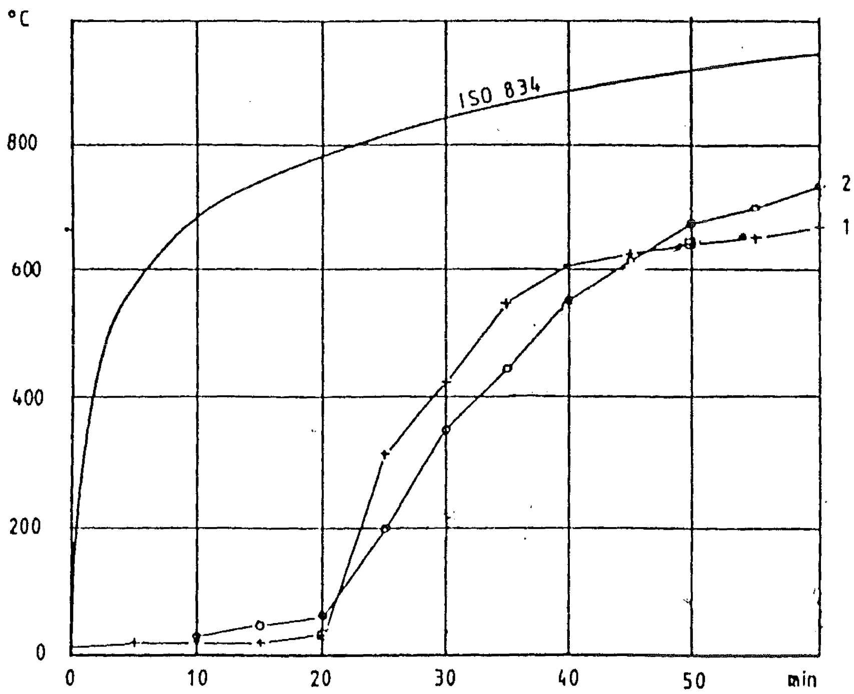


Figure 3.4 Temperature of the screw (curve 1) and at the bearing surface (curve 2) in the fire test of beam-to-column joint.

unit and that no charring occurred at the interface. This finding was somewhat astonishing given the high temperatures measured at the interface. The joint between the screw and the timber was inspected after the test and found to be intact.

British Standard BS 5268 states that a metal connection should be buried into the timber so that it is completely within the residual section as shown in figure 3.5. MP9 (SANZ, 1987) recommends the same guidelines.

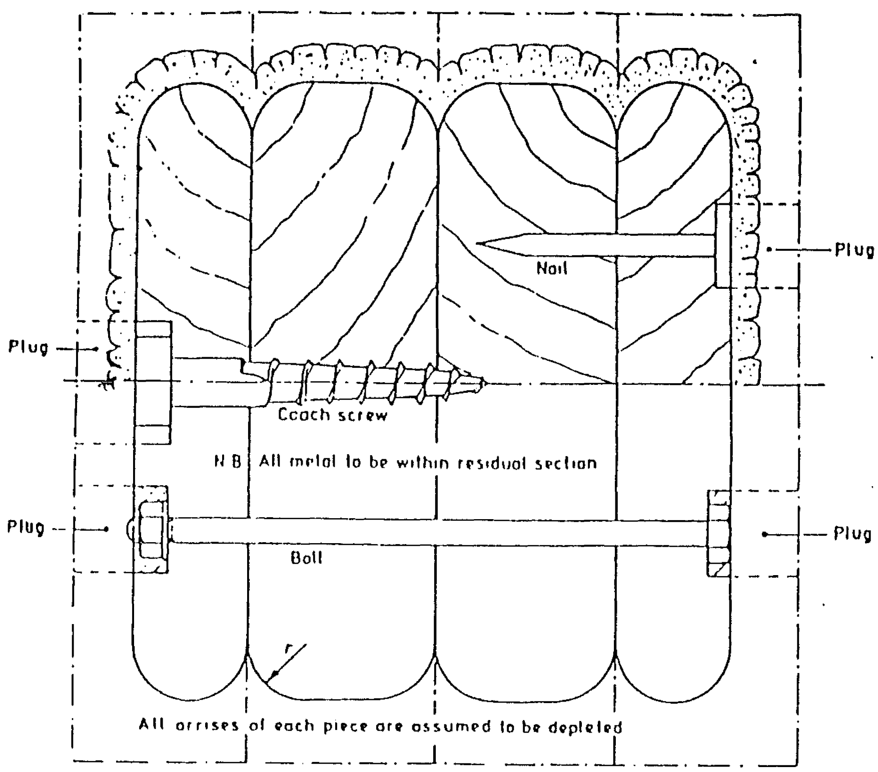


Figure 3.5 Embedding of connection details according to BS 5268.

3.2.2 ENCASING THE JOINT USING TIMBER COMPONENTS

Aarnio and Kallioniemi (1983) have investigated the protection performance of boxes made of glulam timber and particle board. The boxes encased unloaded steel gussets which were nailed to the side of the timber beam. The temperature in the steel gusset was measured during a standard fire. Two different gusset arrangements protected by three different box types were tested. See figure 3.6.

The arrangements were,

(a) Glulam timber box, joined and fastened to the beam with nails

(b) Glulam timber box, joined and fastened to the beam with glue and nails.

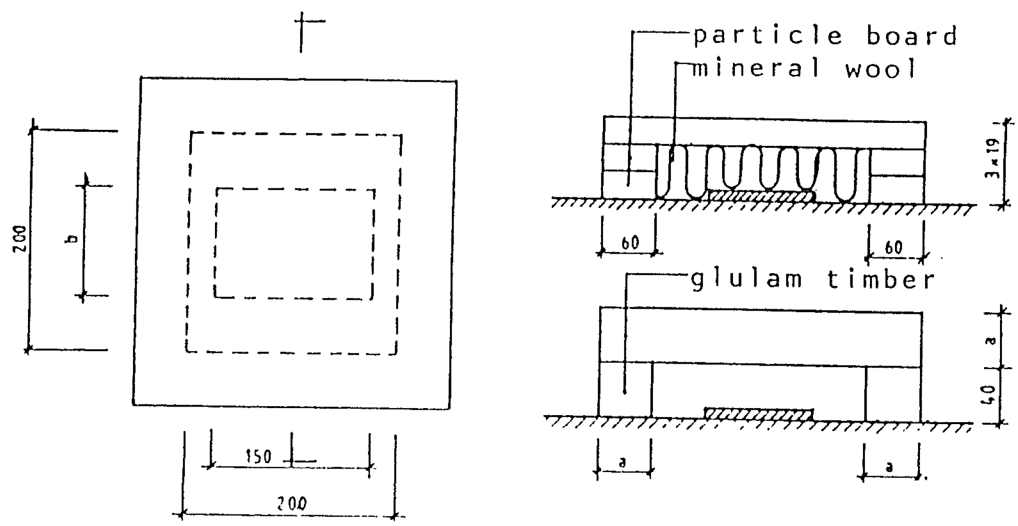


Figure 3.6 Specimen details, gusset and protection arrangements.

(c) Particle board boxes, joined and fastened to the beam with glue and nails, and any internal void filled with mineral wool.

The glulam timber boxes were made from two different thicknesses (40 mm, 70mm) of timber. The results of the experiment are summarised below.

(1) Glulam timber boxes which were glued to the side of the timber, effectively delay the temperature rise in the gusset. After 30 minutes of

standard fire, there was no increase in temperature at all for the box with 70 mm thick timber protection and it was only 15°C in the box with 40 mm thick timber protection.

(2) Particle board boxes filled with mineral wool too, gave reasonably good protection. After 30 minutes of standard fire, the temperature had not exceeded 300°C. Even though the particle board protection was completely charred, the mineral wool still remained in place.

Timber boards for fire protection function by charring and forming an insulating layer of charcoal. King and Yiu (1988) carried out tests on gussets protected by solid timber and exposed to fire on the protected side only. A solid timber of 40 mm thickness was used as protection and its performance was found to be very good. This could be related to the charring and insulating characteristics of solid timber. Considering the dimensions commonly adopted in New Zealand, the researchers recommend a nominal thickness of 45 mm for 1 hour fire resistance.

From the same tests the fire performance of two layers of 18 mm thick construction plywood protection, was found to be inferior. The insulation of the first layer of plywood was found to deteriorate rapidly after 20 minutes of testing with similar deterioration for the second layer after 40 minutes. McNaughton and Harrison (1940) observed that the number of plies had relatively little effect on the resistance to burn through compared with the thickness of the plywood.

3.2.3 USE OF FIRE RESISTANT PLASTERS

Fire resistant plasters are made by bonding together bulk vermiculite or perlite using concrete lime or gypsum. Vermiculite is an expanded mica material and perlite an expanded lava. The density normally varies between 300 to 800 kg/m3 depending on the type of plaster. While plasters with a lower density are relatively soft and suffer risk of mechanical damage, higher density plasters offer hard surfaces and can be used in structures with a high risk of mechanical damage.

Plaster can be sprayed on or applied by hand onto the steel surface. The thickness normally varies between 10 to 40 mm depending on the type of plaster used and the insulation requirements. Maximum thickness for each layer should be kept to 10 to 15 mm before the next layer is applied, to ensure proper adhesion. When certain plasters are used, the surface can be quite rough and it is recommended that putty be used to even the surface. The surface can also be painted.

Rimstad (1979) and Bakke (1978) have obtained temperature results, from 40 minutes of fire testing of 8 and 16 mm thick unloaded steel plates, screwed onto the side of 90 mm wide glulam timber beams. The plates were protected with a 20 mm thick fire resistant plaster. The results are given in figure 3.7 and show that the plaster, in contrast to the fire resistant coatings, is effective at the early stage of the fire giving a temperature delay of about 15 minutes at temperature of about 300 to 500^{o}C.

Non-combustible boards such as gypsum plaster boards with a density of 800 kg/m3 have been used for fire protection of steel construction and they are also suitable for the protection of joints in glulam timber construction. These boards contain water molucules which evaporate around 100^{o}C thus delaying the heat absorption as shown in figure 3.8. Normally 13 mm thick boards are used in one, two or three layers depending on the insulation requirements. Keith(1987) has also recommended the use of 16 mm thick fire rated gypsum plaster board for providing one hour fire resistance. The boards can be fastened onto steel with screws and onto timber using nails or special screws.

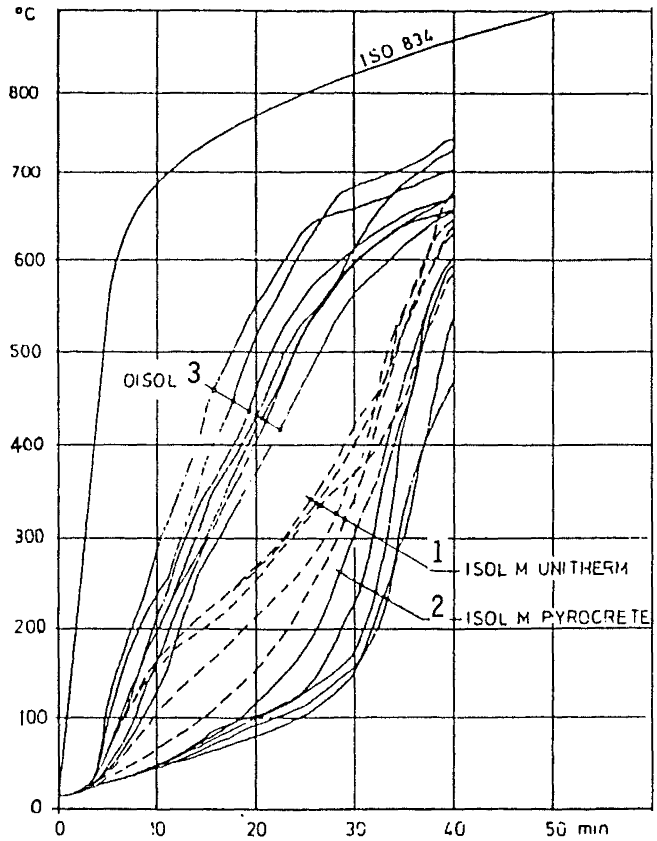


Figure 3.7 Temperature development in steel plate with (1 and 2) and without (3) fire protection. 1 - coating 2 - plaster.

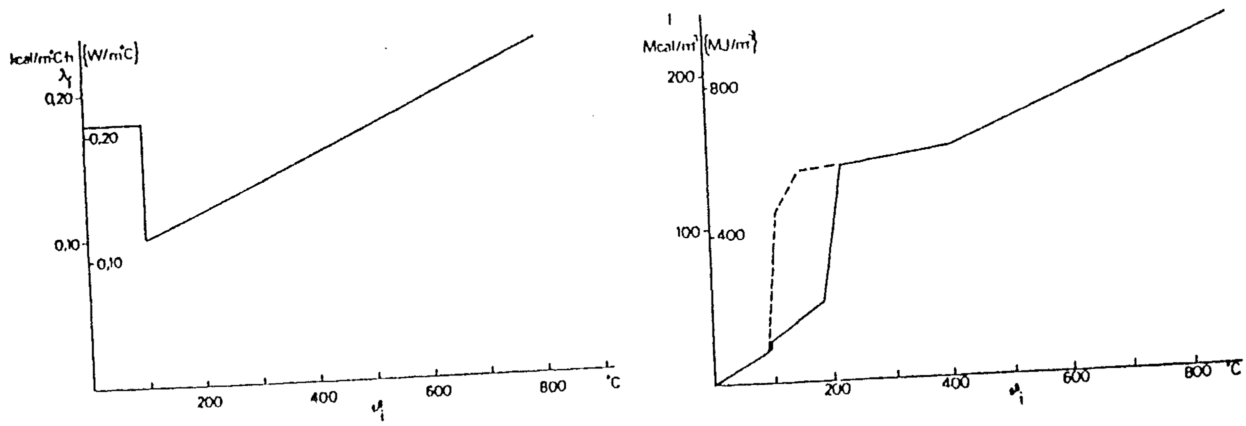


Figure 3.8 Conductivity and heat absorption characteristics as a function of the temperature- gib board

Aarnio and Kallioniemi (1983) have investigated the effect of placing a 5.5 mm asbestos-cement board between the timber and a steel plate of size 75 mm x 125 mm and fire testing the joint for 20 minutes. The joint was unloaded. They found that the asbestos-cement board had reduced the charred depth by 30%.

Mineral wool boards used as protection must have a density of at least 150kg/m3. The thickness of these boards vary between 30 and 70 mm. These can be fastened with nails and special discs to the gusset surfaces. In cases where heavy mechanical damage is possible, special surface protection must be provided. The conductivity and heat absorption characteristics of these boards are indicated in figure 3.9.

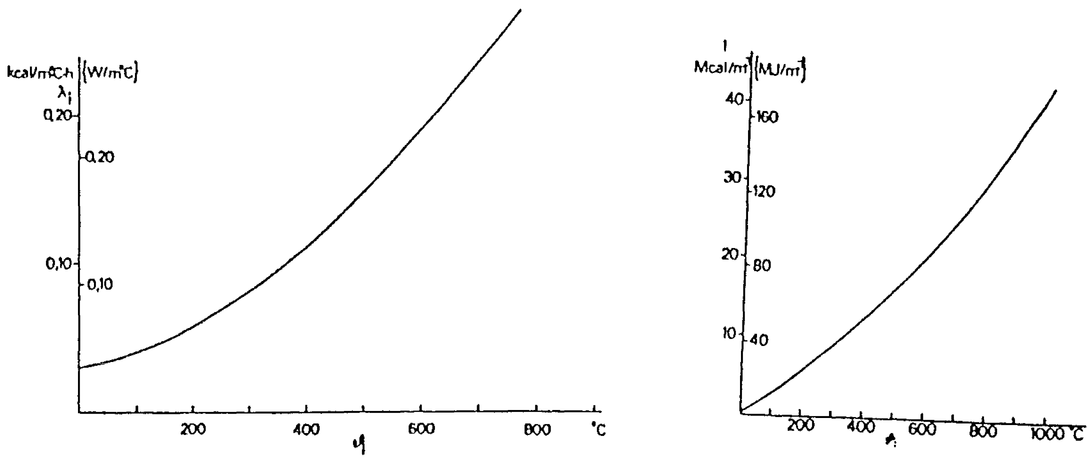


Figure 3.9 Conductivity and heat absorption characteristics in mineral wool boards as a function of the material temperature.

Lihavainen (1976) compared the fire resistance of an unloaded beam-column joint, as illustrated in figure 3.10 with and without a 30 mm thick mineral wool insulation. The temperature development at the gusset/timber contact surface was measured during the test and the results

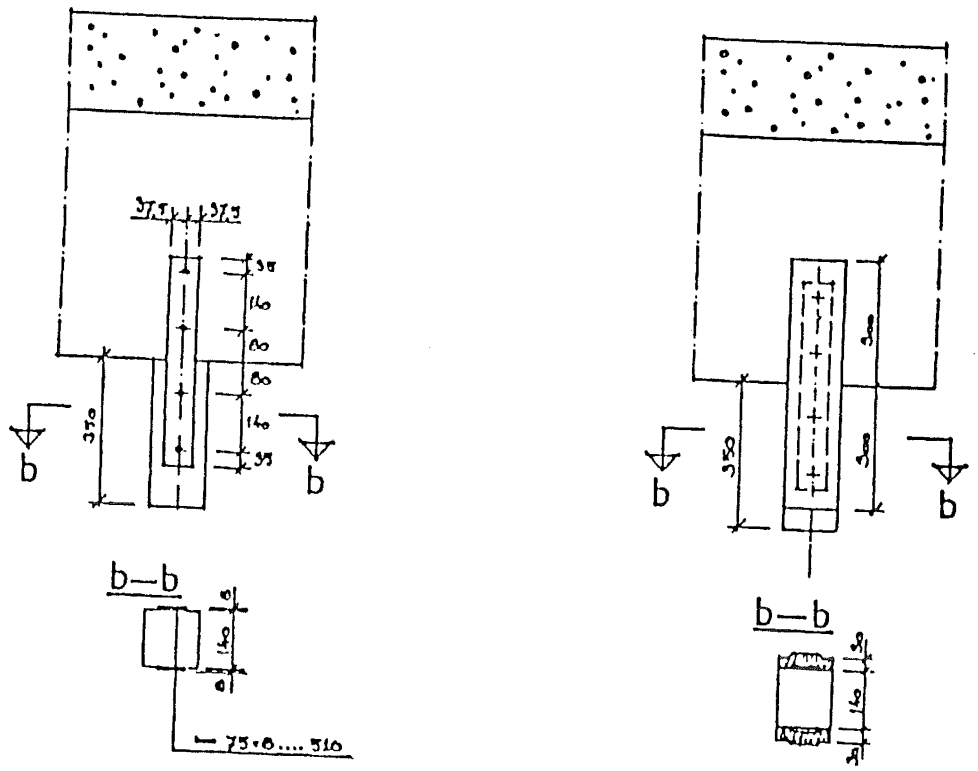


Figure 3.10 Fire tested beam-to-column connections of steel gussets and screws with or without mineralwool insulation.

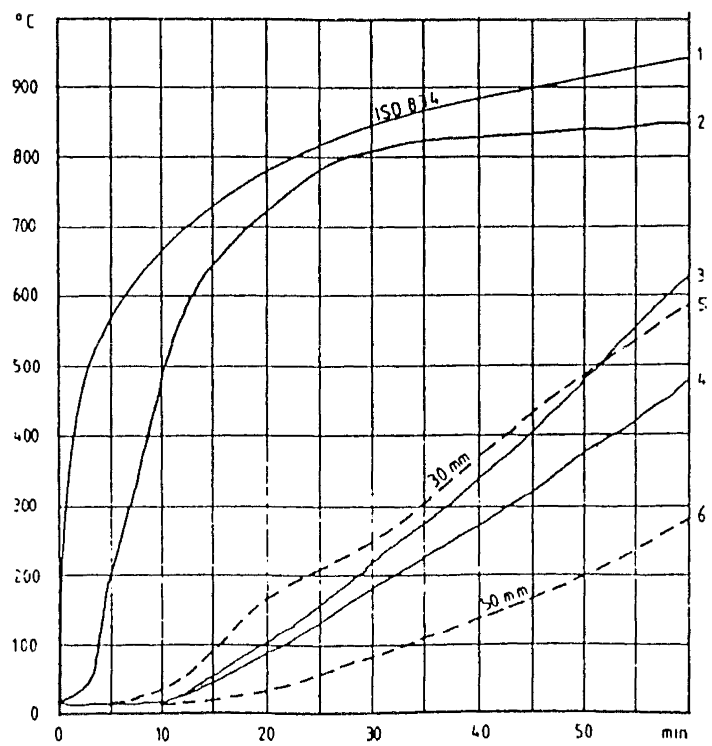


Figure 3.11 Temperature development in steel gusset with and without (curve 2) mineral wool insulation.

are given in figure 3.11. In this figure curve 2 is for gusset without mineral wool protection and curves 3 and 4 are for gusset with protection.

Aarnio and Kallioniemi (1983) have also studied the effects of mineral wool insulation on the temperature development of 75 and 125 mm wide flat plates and the charring of the timber around these plates. The dimensions of their test samples are given in figure 3.12. The mineral wool is 30 or 50 mm thick with a density of 150kg/m3. They were fastened with nails and glued to the timber surface. No forces other than self-weight was applied between the plate and the timber. The results of the temperature measurements are given in figure 3.11 (curves 5 and 6). The extent of the charred zone after 60 minutes of fire is shown in fig.3.13.

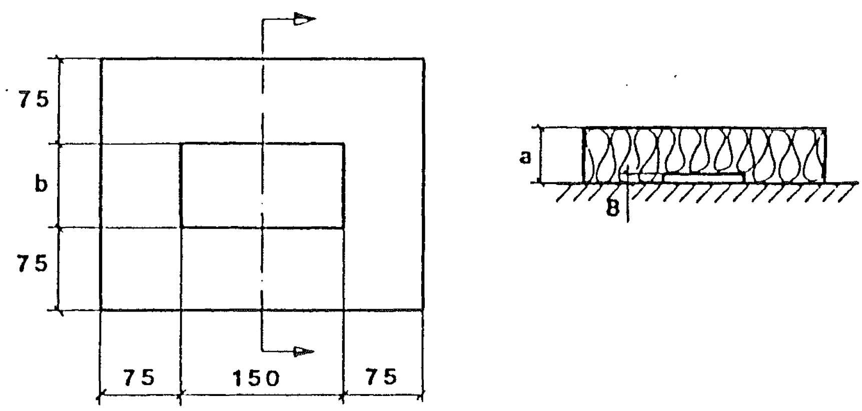


Figure 3.12 Steel plate insulated with mineral wool as tested by Aarnio and Kallioniemi (1983) a = 30 or 50 mm and b = 75 or 125 mm

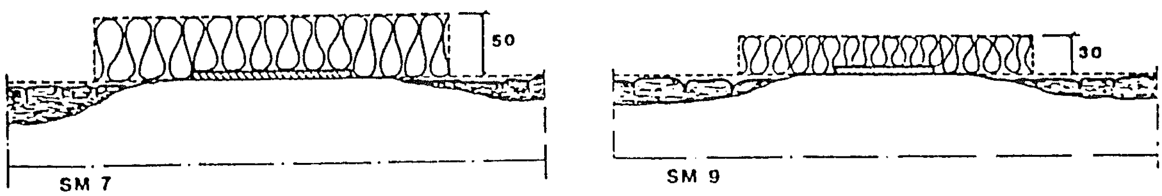


Figure 3.13 Extent of the charred zone after 60 minutes of standard fire

Sprayed mineral wool consists of mineral wool fibres mixed with concrete or cement as a bonding agent. This mixture together with water is sprayed in a fine particle form to the desirable thickness directly onto the surface which is to be insulated. The thickness of the insulation is normally between 10 to 30 mm depending on the insulation capacity required. The insulation is relatively soft and this has to be taken into account in applications where there is a risk of direct mechanical damage. Improved mechanical strength can be achieved with the inclusion of glass fibre materials (Magnusson, Pettersson and Thor, 1974)

Rimstad (1979) and Bakke (1978) have shown temperature results from 40 minutes of fire testing of 8 and 16 mm thick unloaded steel plates screwed onto the side of 90 mm wide glulam timber beams and coated with three layers of 'unitherm' fire resistant coating. It was observed that the coating had no effect until the temperature reached beyond 200^{o}C and the delay in temperature rise is about 10 minutes at temperatures of 300-500^{o}C. Rimstad used the attainment of 300^{o}C at the surface between the steel plate and the timber as his failure criterion and gave the time to failure as 28 minutes.

3.2.4 APPLICATION OF INTUMESCENT COATINGS

Special paints such as Nullifire S60 have achieved fire resistance ratings of upto 1 hour for protection of structural steel members. These intumescent paints can also be used to protect steel gusset arrangements. At elevated temperatures (150-200^{o}C) these coatings swell significantly and form an insulating foam. Generally this type of protection for steel surfaces loses favour when the section factor(heated perimeter/area) is very high. From the tests conducted by King and Yiu (1988) this type of protection was found to be unsuitable as an independent protection for the large steel gussets commonly used in New Zealand due to faster heating up, than for structural steel members.

Similar formulations which are being developed for timber surfaces (such as Nullifire WD) can delay the amount of char, but such products have not yet received fire resistance ratings in New Zealand, and no studies of their use on connections are known of.

The better performance of these coatings involves a greater number of applications and a specific period between each application, which may not be attractive in terms of application costs. Strict quality control is also required. Further, these coatings can not be used in an external environment.

3.3 DETAILING OF PROTECTION

The importance of good detailing in protection systems can not be overemphasized. The principal objective in designing fire protection systems is to achieve smooth, flat, unbroken surfaces with joints carefully detailed and fabricated to fit closely. Fire attacks thin sections and sharp corners much more readily than flat, smooth surfaces. Cracks, gaps and concealed openings encourage an increased rate of destruction. For this reason, the performance of nail-laminated members is inferior to glue laminated members.

The flue action facilitated by gaps or cracks in the protections can be avoided by filling them with fire resistant glue or intumescent paint (Baber and Fowkes, 1984; Smith, 1985 ; Tan, 1988). Another problem that can be overlooked in the arrangement of protections is the reduction in edge and end distances of the outermost nails. This can happen as a result of charring either in the main member or the plywood gusset, if the protection is provided on the gusset surface only. The problem is aggravated by the fact that these nails are the most highly stressed. Hence, it becomes necessary to protect the gusset as well as the connected members on all sides. The manner of securing the protection to the joint also plays a vital role in offering good fire protection (Golding, 1984).

CHAPTER FOUR

BEHAVIOUR OF NAILS AT ELEVATED TEMPERATURES

4.1 INTRODUCTION

The behaviour of steel nails at elevated temperatures, subject to the loading and arrangement similar to that in a gusset connection has not been studied in depth. However, it has been shown that the relative size of the nail and its embedment within the timber provides superior performance to that of other fasteners. As nails are thin compared with their length, heat is not easily transferred along them to the stressed portions, unlike bolts or toothed plates (Leicester et al 1979).

4.2 TYPES OF NAILED JOINTS

Both nailed steel and plywood gusset connections are widely used for the knee and apex joints in portal frames made with glulam timber. Such joints may become common in multistorey timber construction too. In the case of steel gusseted joints, the nails are driven through predrilled holes in the gusset plate. With plywood gussets they are driven pneumatically. The length of these nails can vary. In a fire environment, longer nails char the adjacent timber less, and thus perform better than the shorter ones (Aarnio and Kallioniemi, 1983).

4.3 THE CHARRING CHARACTERISTICS OF TIMBER AROUND HOT NAILS

Investigations conducted by Aarnio (1979) indicate the charring characteristics of timber around unloaded nails. Both long and short nails of different diameters were used. Minimum spacings as per code requirement were adopted between the nails. After 35 minutes of exposure to the standard fire it was found that the charred zone had been deepened 20-30 mm in the immediate vicinity of the nails. The charred zone had penetrated deeper around bigger nails. Between the nails the charred zone had penetrated 5-10 mm deeper than in the sections without nails. Penetration of the charred zone was deeper between smaller nails as shown in figure 4.1.

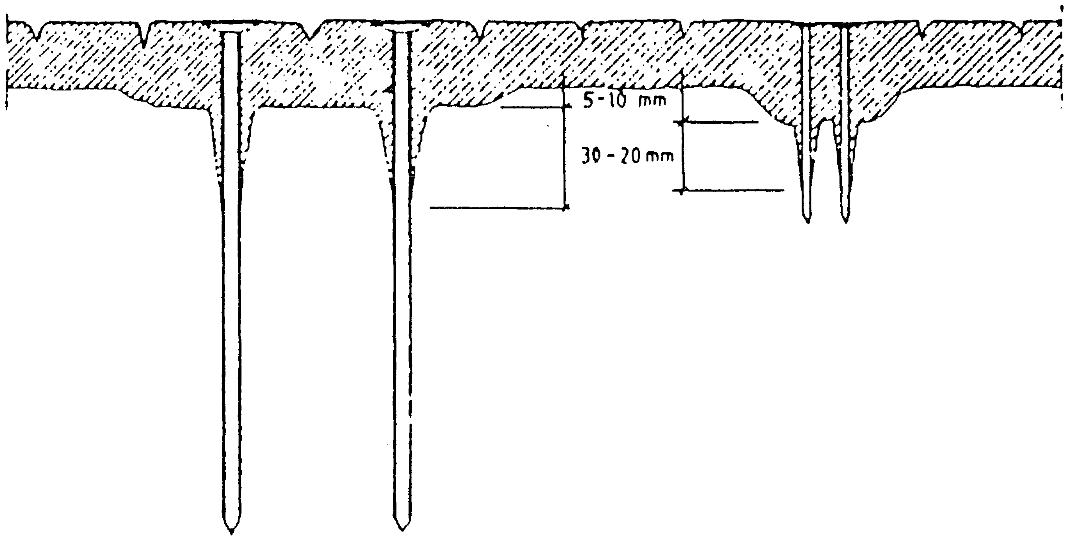


Figure 4.1 Schematic diagram of the penetration of the charred zone in the vicinity of large and small nails.

This could be due to the heat capacity of the nails. For a given fire exposure the heat capacity of smaller nails being smaller, more heat is conducted into the wood, thus charring more. This phenomenon is also confirmed by the tests carried out by King and Yiu (1988).

Aarnio and Kallioniemi (1983) also show the results from similar tests using 5.1 mm diameter nails of different lengths (115 to 145 mm) and 70 minutes of exposure to the standard fire. They observed a 10 to 20 mm increase in the penetration of the charred zone on the side from which the nails were put in. However, by the side of the nails the penetration increased to about 60 mm, irrespective of the nail lengths. On the opposite side, the tips of the longest nails were found exposed at the end of the test. Also the charred zone around them had diminished. The apparent reason for this was given as the conducting away of heat which reduced the amount of heat transmitted to the timber around the nail.

4.4 TEMPERATURE DEVELOPMENT ALONG THE LENGTH OF THE NAIL

The temperature development along the length of the nail was one of the aspects studied in the tests conducted at the Building Research Association of New Zealand (King and Yiu 1988).

Two different nail lengths (45 mm and 60 mm) with different spacings were used and the temperatures along the shorter nails were higher irrespective of the spacing. Significant differences in temperatures were observed between the nail head and the nail tip, indicating the existence of a thermal gradient along the nail. At the end of the test, the temperature readings implied a linear relationship with the length of the nails as shown in figure 4.2, even at different locations and spacings. This was related to the quasi-linear relationship between temperature and depth along charred timber in the range of 200 to 850^{o}C as shown in figure 4.3.

Tests conducted by Swane and Vagholkar (1968) also confirm the existence of a linear thermal gradient having consistent temperature values at each location as shown in figure 4.4. They also substantiated the fact that the heat transfer along an embedded nail shank is proportional to the volume of the embedded metal.

Tests were also carried out, as part of this study, to determine the variation of temperature along the length of the nail. The existence of a thermal gradient along the length of the nails, exposed to fire is fur-

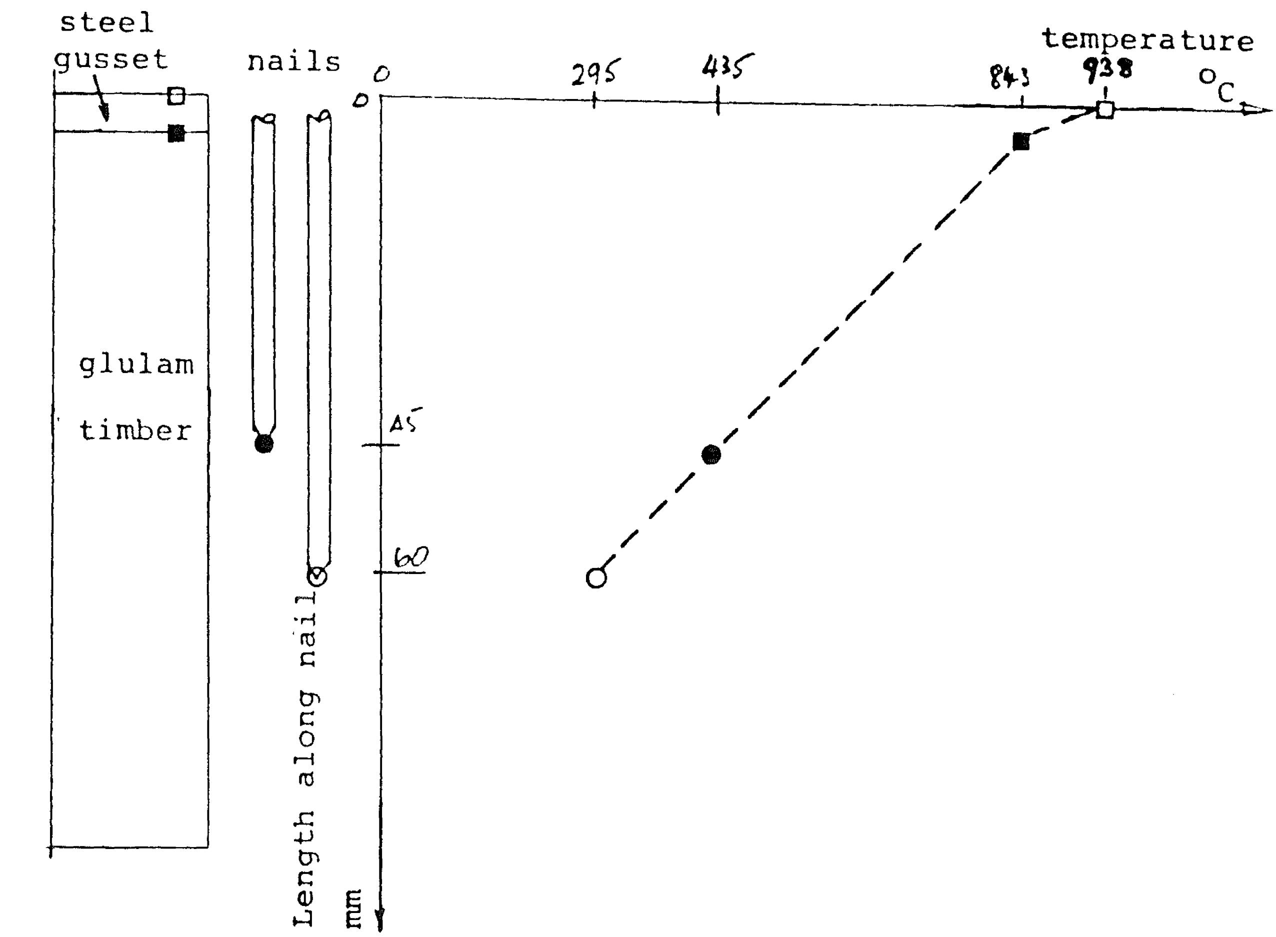


Figure 4.2 Temperature development in nails

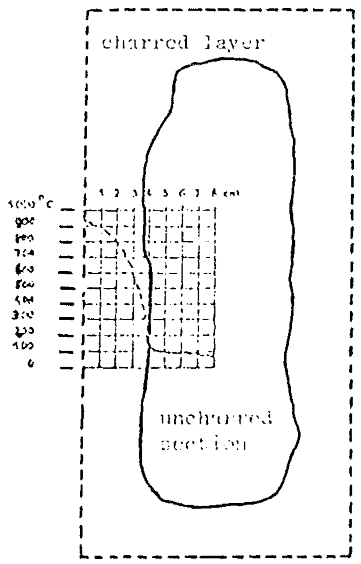


Figure 4.3 Temperature observed in the section of 160 x 360 mm timber beam after 60 minutes of fire exposure (Sauvage 1985)

ther verified in these tests. From the above findings it can be concluded

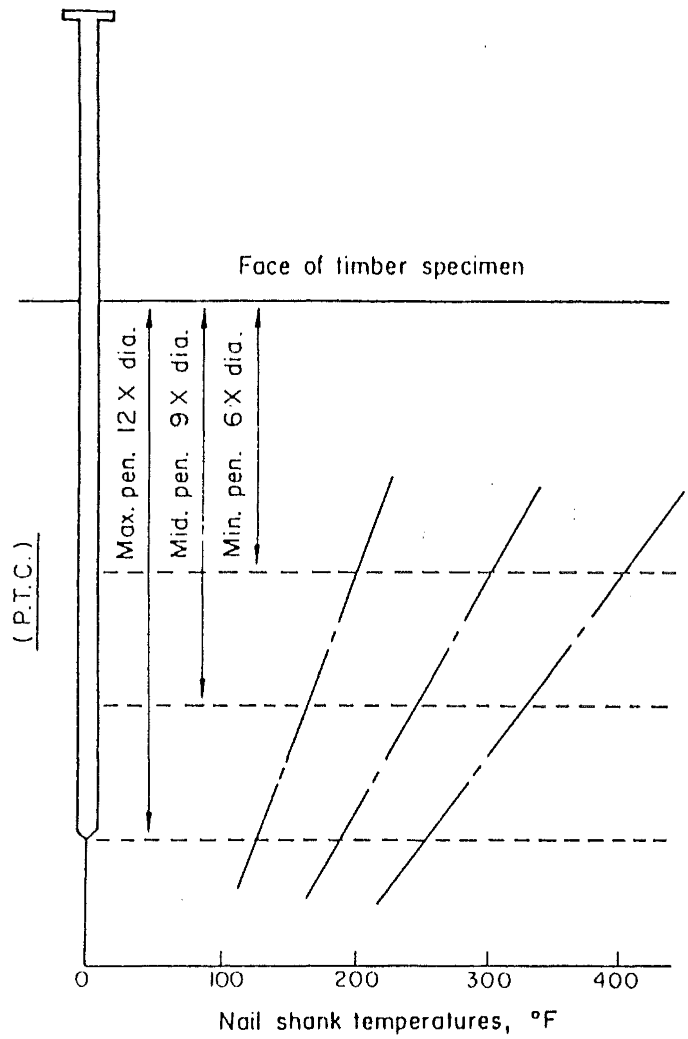


Figure 4.4 Nail shank temperature gradient graph

that

(1) There exists a temperature gradient, along the length of the nail which is exposed to a fire.

(2) The length of the nail plays an important role in absorbing and dissipating heat. The amount dissipated is greater for shorter nails, leading to greater charring around the immediate vicinity of the nail.

(3) Nails used in a fire environment should be long enough to ensure less heat dissipation and thus continued rigidity of the joint.

4.5 WITHDRAWAL CHARACTERISTICS OF NAILS AT ELEVATED TEMPERATURES

Withdrawal characteristics of nails at elevated temperatures play an important role in the fire performance of the joint. When loaded and exposed to fire, the probability of failure of a joint is increased if the nails are shorter, than if they are longer. For this reason, the nails used in gusseted joints must at least be of a certain minimum length.

Anderberg (1980) has examined the withdrawal resistance of different types of nails after 15, 30 and 60 minutes of exposure to the standard fire. The results indicate that the withdrawal resistance besides being related to the type of nails, also depends on the anchorage length and the maximum temperature at the tip of the nail.

Swane and Vagholkar (1968) have also conducted tests to determine the withdrawal resistance of plain shank nails. In order to establish a statistical relationship between withdrawl resistance and diameter, depth of penetration of the nail, they used nails of three different diameters (3.6 mm, 2.59 mm and 2 mm) with three different penetration depths (6, 9 and 12 times respective diameters). The effect of temperature on withdrawl resistance observed in these tests is indicated in figure 4.5. Further, they established a linear relationship between the logarithm of withdrawal resistance and the logrithm of diameter and the depth of penetration from these studies. The two destructive factors indicated by them as being associated with the loss of resistance between the nail shank and the timber fibres were,

(1) Shrinkage of the timber fibres immediately surrounding the nail

(2) Charring of these timber fibres.

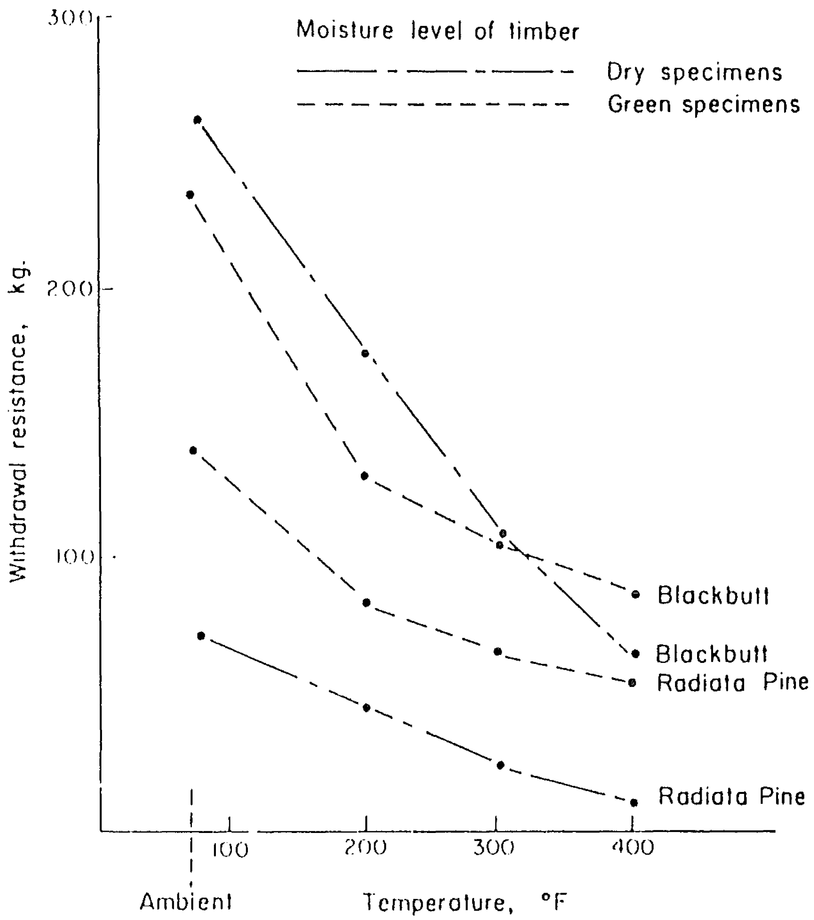


Figure 4.5 Plot of withdrawl resistance against temperature

It is the interaction between the density and moisture content of timber and temperature variables that establish the relative destructiveness on resistance of the above two factors. While higher density and lower moisture content of timber increase the withdrawal resistance, increased nail shank temperature decreases it.

CHAPTER FIVE

EXPERIMENTAL INVESTIGATIONS OF NAILED-ON GUSSETED JOINTS

5.1 INTRODUCTION

The experiments described here were intended to simulate the standard fire tests (ISO 834) carried out in a pilot furnace at the Buiding Research Association of New Zealand (BRANZ). In the BRANZ study, gussets of both steel and plywood were nailed to blocks of glulam timber, and covered with various forms of protection. The specimens were unloaded and exposed to fire on one side only. Temperatures were measured behind the protection and behind the gussets for a 1 hour period. These temperature results shown in figure 5.1 provided the input information for the experiments described in this report.

The experimental program, the characteristics of test materials, the apparatus used and instrumentation are described first, followed by a more detailed description of the tests.

5.2 THE EXPERIMENTAL PROGRAM

The experiments carried out in this project can be grouped under six sections. They are:

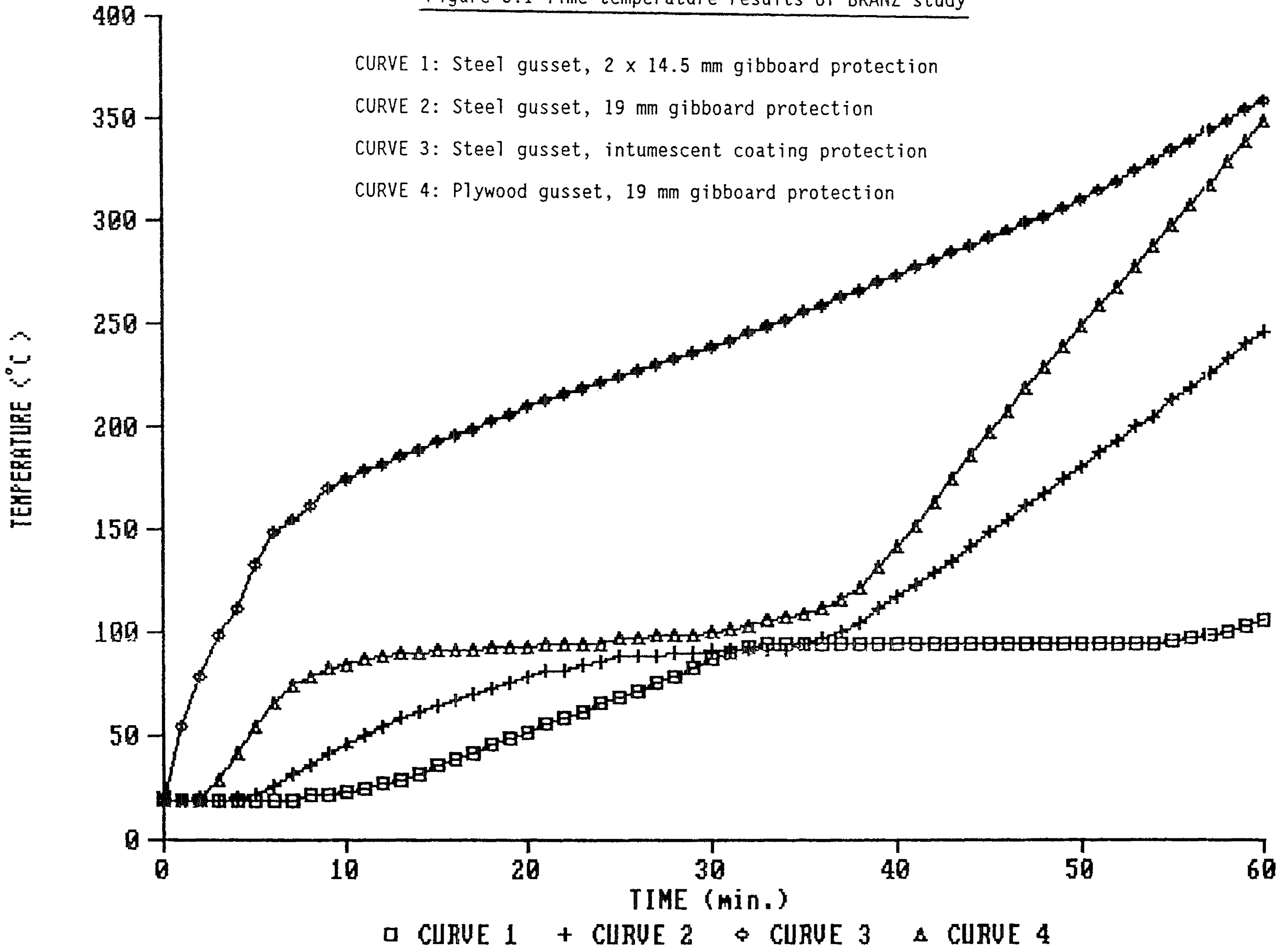
(1) Preliminary tests simulating time-temperature relations obtained during the pilot tests at BRANZ.

(2) Tests to determine the load-slip characteristics of the joints under cold conditions

(3) Tests on steel gusseted joints to determine the load-slip characteristics under fire conditions

(4) Tests on plywood gusseted joints to determine the load-slip characteristics under fire conditions

Figure 5.1 Time-temperature results of BRANZ study



(5) Tests to determine the temperature development along the length of the embedded nails.

(6) Tests to determine the bending strength of nails

5.3 THE CHARACTERISTICS OF TEST MATERIALS

The characteristics of the joint component materials used in these tests are described in the following paragraphs.

5.3.1 GLULAM TIMBER

The timber used in these tests was radiata pine, glue laminated timber supplied by Hunter Timbers Ltd, Nelson. All of the specimens were cut from one glue laminated beam 500 x 90 mm x 10 m long, which was made from the same laminating stock used for the subsequent full scale BRANZ tests. The beam was ordered as No 1 framing grade, although there were very few defects close to the grade limits. The timber had a density of 450-550 kg/m3 and a moisture content averaging 14%. The glue used in bonding the laminations was melamine urea. The dimensions of the loading frame, the furnace and the direction of loading relative to the grain direction determined the sizes of the timber to be used in these tests.

5.3.2 STEEL GUSSET

5 mm thick mild steel plates of size 200x400 mm were initially used as steel gussets. The area of the gusset that was exposed to fire had a size of only 200x200 mm and this resulted in heat being dissipated from the exposed part to the unexposed part. To reduce this heat loss, the size of the steel plates were reduced to the size shown in figure 5.2. The plates were predrilled with six holes to receive the nails and five other holes to attach the thermocouples. A larger hole for receiving the

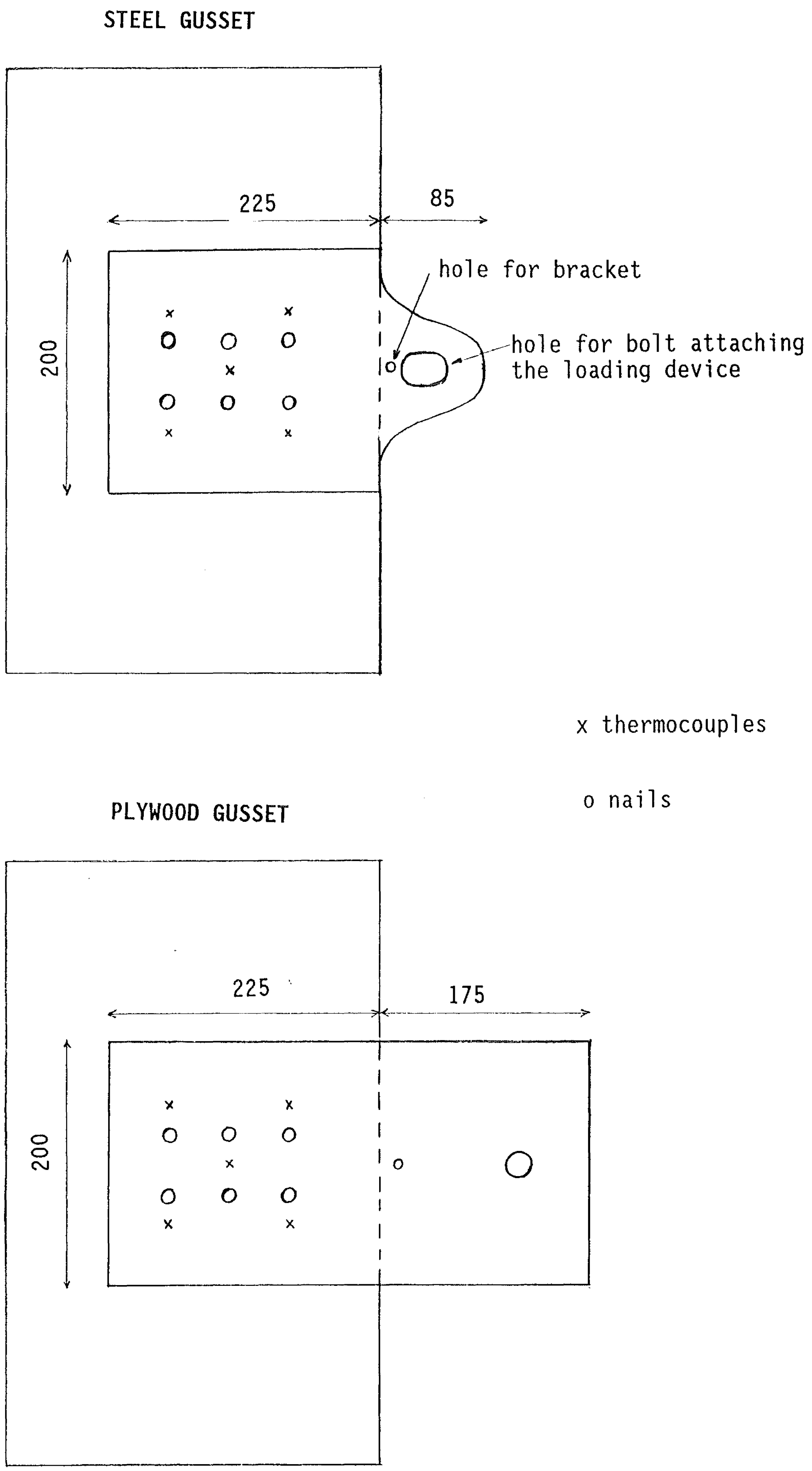


Figure 5.2 Gusset geometry and locations of nails and thermocouples

bolt attaching the loading device and another for attaching the bracket for displacement measurement were also made.

5.3.3 PLYWOOD GUSSETS

18 mm thick construction plywood blocks having a size of 200x400 mm were used as the plywood gussets. The individual pieces were all cut from a bigger piece and then pierced with holes for receiving the bolt attaching the loading device as shown in figure 5.2. The plywood was Cp-D grade radiata pine plywood. The face grain was perpendicular to the direction of loading.

5.3.4 NAILS

For the steel gussets, 75 mm long plain shank nails, of nominal diameter 3.15 mm were used. These nails had an ultimate strength in excess of 700 Mpa. In the case of plywood gussets, 85 mm long gun nails of diameter 3.33 mm were used which had an average ultimate strength of 711 Mpa. A random sampling of the diameters of the plain shank nails revealed a large variation and the average nail diameter was found to be 3.23 mm as against the specified 3.15 mm. This discrepancy did not occur with gun nails. Nails of 3.15 mm diameter and of shorter lengths (40 mm and 30 mm) were also used with steel gussets to study their effect on the joint performance. Nails of 4.35 mm diameter and of 45 mm length were also included in one of the tests. The shorter nails were all galvanised, because that was the only source of supply.

5.3.5 TYPES OF PROTECTION

Three different types of protection were assumed in the tests. They were:

(1) Two layers of 14.5 mm thick 'FYRESTOP GIBRALTAR BOARD'

(2) One layer of 19 mm thick 'FYRELINE GIBRALTAR BOARD'

(3) Intumescent coating 'NULLIFIRE S60'

The first two are referred as' gibboard protection' and the third one as 'intumescent coating' throughout this report.

5.4 DESIGN OF TEST APPARATUS

All testing was carried out in an Instron universal testing machine in the Model Structures Laboratory at the University of Canterbury. As the aim of these tests was to determine the load-slip characteristics of the joint when exposed to fire, three things needed careful consideration. They are:

(1) A way of mounting the specimen (glulaminated timber block) on to the bed of Instron test machine, so that load can be applied.

(2) A means of applying load to the gusset, nailed to the timber block.

(3) A means of simulating relevant thermal exposure.

The details of the above three aspects are explained briefly.

5.4.1 ARRANGEMENT FOR MOUNTING THE JOINT

It was considered important to have the gusset plates in a horizontal orientation to ensure uniform heating. Hence the mounting arrangement consisted of two steel I-beams, two angles and a square steel plate with the bottom flanges of the I-beams welded at one end to the steel plate which was bolted to the bed of the Instron test machine. The angles which acted as buttresses against which the timber specimens rested, were welded to the top flanges of the I-beams at the other end. Figure 5.3 shows the the arrangement.

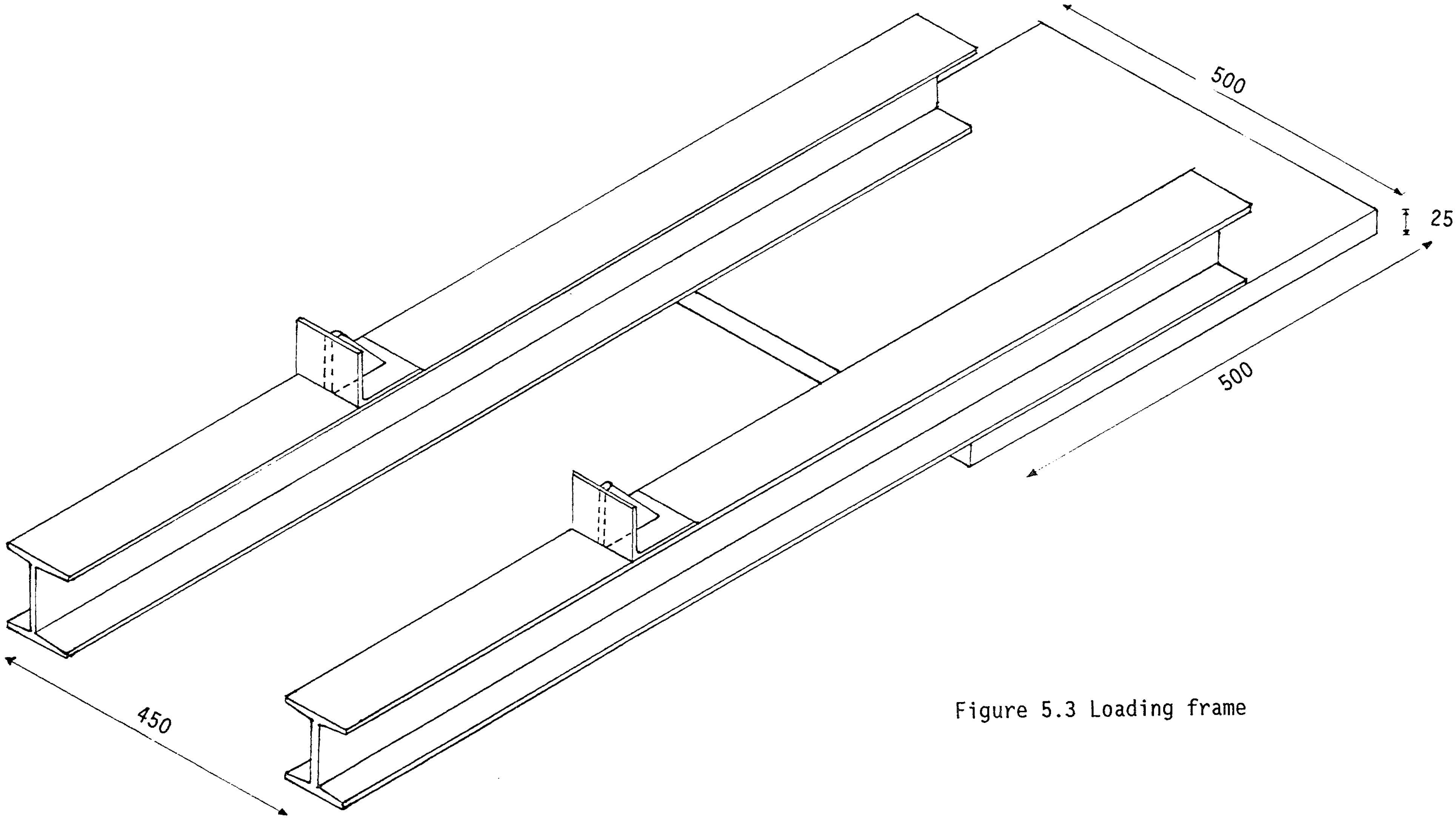


Figure 5.3 Loading frame

5.4.2 LOAD TRANSMISSION TO THE JOINT

A steel wire rope was used to transmit the load from the Instron test machine to the joint. The wire rope converted the vertical load application of the test machine to horizontal shear load input to the joint, by passing through a pulley as indicated in figure 5.6. The rope was shackled at one end to the gusset and at the other to the Instron head.

5.4.3 A DEVICE FOR SIMULATING FIRE CONDITIONS

The maximum temperatures reached behind the protections and behind the gussets as recorded during the BRANZ tests were found to be well below 500^{o}C. Thus a small portable furnace of 500^{o}C capacity appeared sufficient for the tests. The furnace as fabricated contained an electrically heated domestic stove element housed inside a two-skinned rectangular sheet metal box, the periphery of which was filled with mineral wool fibre insulating material. The furnace had metal flanges so it could be clamped to the timber test specimen. The furnace was run by a thermostat control box, having a graduated dial. Figure 5.4 shows the complete setup.

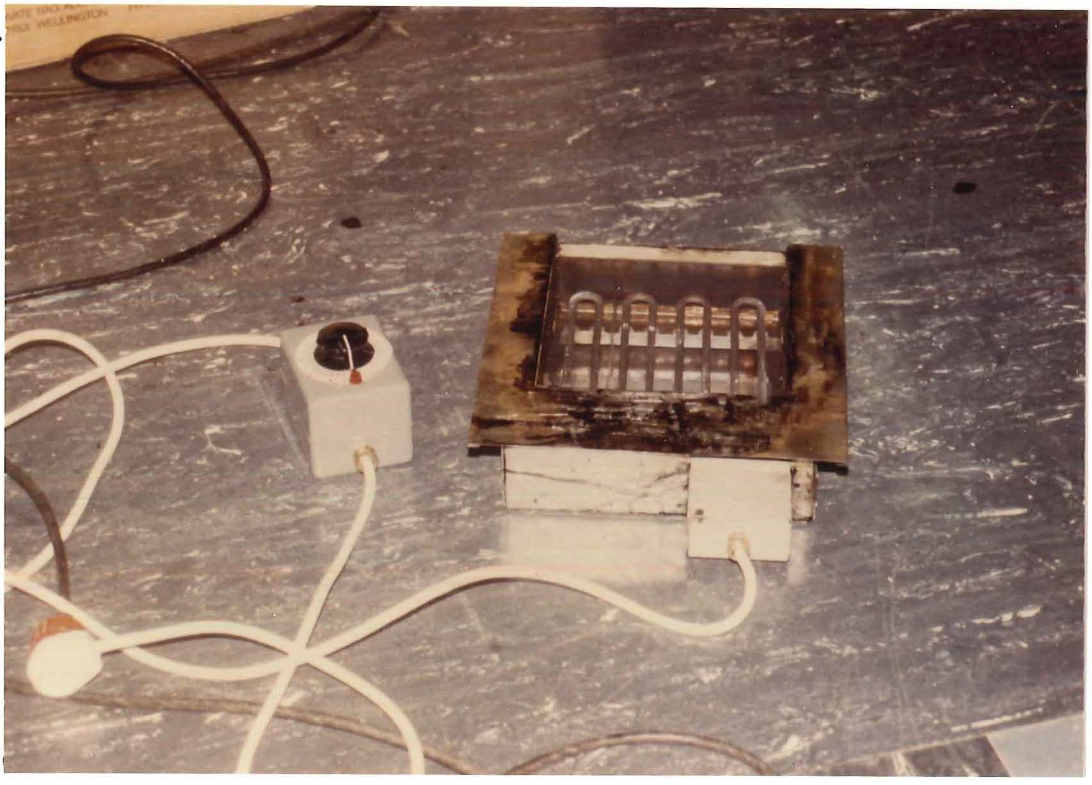


Figure 5.4 Furnace used for the tests

5.5 INSTRUMENTATION

A comprehensive instrumentation procedure was established to trace the complete behaviour of the joint during the course of the test. The load applied, the displacement of the joint and the heat input to the joint were all monitored and recorded at regular time intervals. A computer datalogging system proved most useful in this process. The relevant aspects of these are detailed in the following paragraphs.

5.5.1 TEMPERATURE MEASUREMENT

To measure the temperatures existing within the joint, five iron-constantine thermocouples were used. The location of these thermocouples for steel and plywood gussets are indicated in figure 5.2. These are screwed on to the gusset surfaces using short screws with washers which ensured tight fit and thus a good contact. The other ends of the thermocouples were connected to a computer controlled datalogging system which sampled the temperatures at twenty second intervals. In the case of steel gussets the temperatures on the glulam surface for various protections as provided by the BRANZ tests were followed. This was achieved by simply maintaing the heat input to the steel plate in such a way that the temperatures measured on the inner surface at any time corresponded to the temperatures at the same location in the BRANZ tests. For plywood gussets the temperatures behind the protection on the outer surface of the gusset as provided by the BRANZ tests were followed. This was succesfully simulated in the laboratory by placing a thin layer of fibrous plaster board (8 mm) on the gusset surface and maintaining a reduced heat input. The complete details of temperature measurement for both the gussets are shown pictorially in figure 5.5.

The temperatures indicated by the thermocouples at different loca-

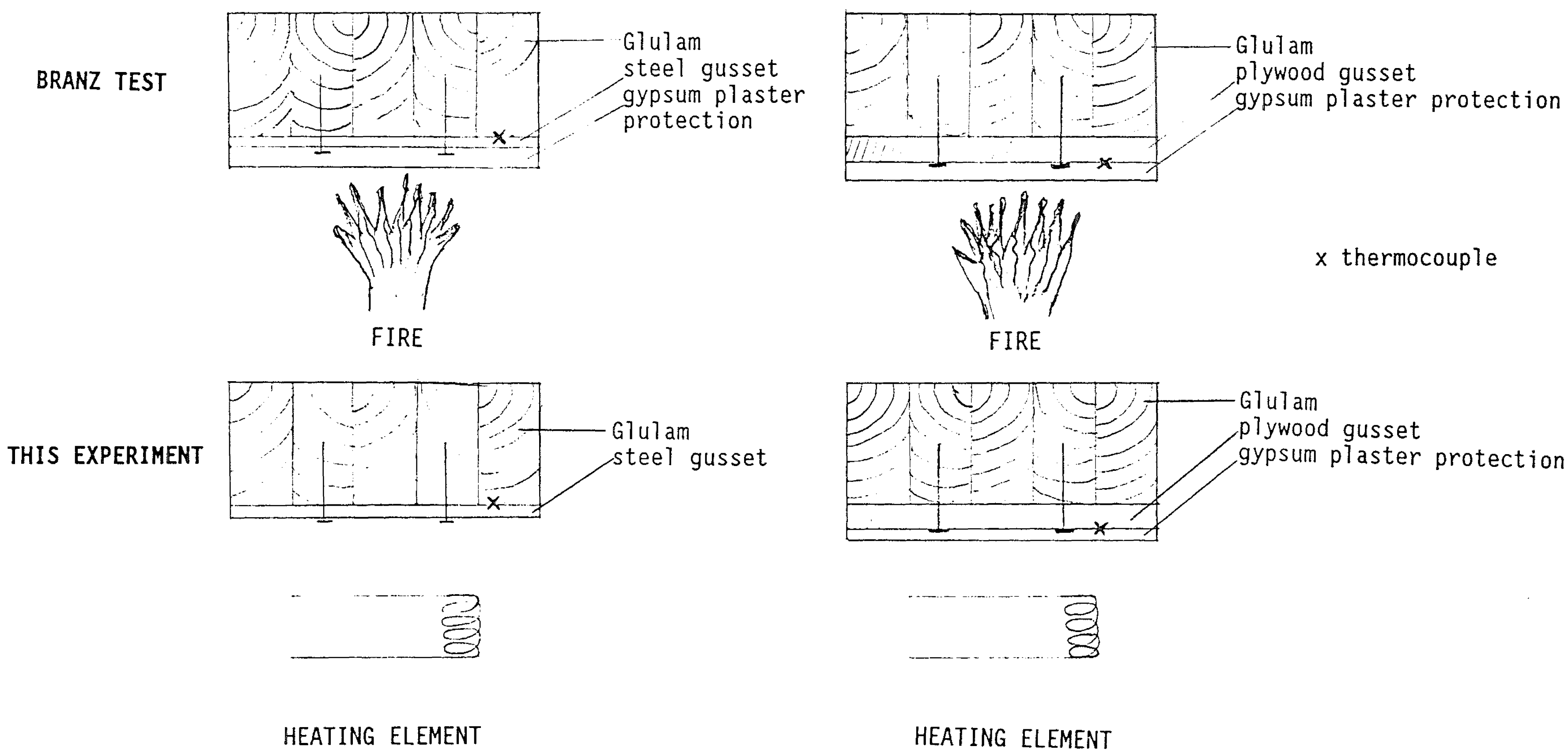


Figure 5.5 Details of temperature measurement and other test conditions

tions varied to some extent. The thermocouples located closer to the loading point in the gussets generally registered slightly lower temperatures than the rest. This is because the portion of the gusset around the loading point being unexposed to fire, received heat from the heated portion, forming a thermal gradient. Therefore it was decided to take the average of the temperatures indicated by all five thermocouples as the test temperature.

Heat input to the furnace was achieved through a thermostat box, having a knob and a graduated dial. The graduations corresponded to specific heat input and by adjusting the knob, the required amount of heat could be maintained at any time.

5.5.2 LOAD MEASUREMENT

Load was measured through a load cell calibrated to the required range and attached to the head of the test machine. Maintaining a constant load, as the joint underwent thermal exposure was done manually.

5.5.3 DISPLACEMENT MEASUREMENT

Joint displacement was measured using a linear potentiometer having a possible displacement range of up to 30 mm. The displacement of the joint was sampled every two to three seconds and displayed on the screen. In order to verify the readings indicated by the linear potentiometer, a dial gauge was also used. A steel bracket attached to the gusset as shown in figure 5.6 acted as the reference point against which the needles of both the displacement gauges rested. Generally, good agreement was observed between the displacements indicated by both the displacement gauges.

5.5.4 DATALOGGING AND DATAPROCESSING SYSTEMS

A computer datalogging system developed in the department using ASYST computer software was used for temperature and displacement data acquisition. The data were sampled every twenty seconds and displayed on the screen. It was also possible to record the data in the hard disk memory at user defined time intervals.

Dataprocessing was carried out as soon as a test was over, by retrieving the data from the hard disk memory and processing it using VPP computer software package.

5.6 DETAILS OF TESTS

This section describes the details of all six sections mentioned under section 5.2. Generally, the aim of the tests, specimen preparation, loading details, test procedure and observations made during the tests are discussed.

5.6.1 PRELIMINARY TESTS SIMULATING TIME-TEMPERATURE RELATIONS

The principal aim of these tests was to simulate the time-temperature relations obtained for various protections during the BRANZ study and learn to operate the equipment. Discarded glulam timber offcuts were used for these tests. The sizes of these timber blocks varied, but they were usually of 600 x 450 x 100 mm. The thermocouples were first fixed to the steel or plywood gusset. The gusset was then nailed on to the timber block and the timber block was transferred to the loading frame and aligned properly. Next the furnace was clamped to it.

The experiment was started by switching on the thermostat control to start the furnace and simultaneously starting the computer and a stop watch to record the data and to track time respectively.

A number of trials were found necessary for each temperature profile before good control over heat input could be established. The position of the thermostat control knob and the temperature that existed in the joint at a particular time were written down. This helped in adjusting the thermostat to input the required amount of heat.

Generally a time lag was observed between adjusting the thermostat control knob and any change in temperature readings. This time lag was much less for steel gussets than for plywood gussets. Thus a better accuracy of simulation of time-temperature relations was obtained for steel gussets with various forms of protection, than for plywood gussets.

5.6.2 TESTS FOR LOAD-SLIP CHARACTERISTICS OF NAILS UNDER COLD CONDITIONS

These tests were conducted, to get an idea about the load-slip behaviour of the nails used, so that their performance under fire conditions could be compared. For the steel gussets, another aim was to study the effect of the manner of nail driving on joint performance. The complete details of the tests carried out are presented in table 5.1.

Once the effect of the manner of nail driving was known from the initial tests, all further tests were carried out having the nail heads driven loose. i.e., without any further nail driving once the nail head came flush with the gusset surface. This is consistent with real situations because any prestressing of the nails due to hard nail driving could be expected to relax with time.

Once the gusset was attached to the timber specimen, it was placed on the loading frame and the loading wire bolted to the gusset hole. The linear potentiometer was attached to the top timber surface with its needle resting against the reference bracket which was attached to the

TABLE 5.1 : DETAILS OF TESTS

Timber specimen size	Gusset details	Nail details length - dia. mm mm		Loading direction relative to grain
495x350x90 mm	5mm steel	75	3.15	parallel
495x1000x90 mm	5mm steel	75	3.15	perpendicular
495x350x90 mm	5mm steel	40	3.15	parallel
495x1000x90 mm	5mm steel	40	3.15	perpendicular
495x350x90 mm	5mm steel	30	3.15	parallel
495x1000x90 mm	5mm steel	30	3.15	perpendicular
495x350x90 mm	18mm plywood	85	3.30	parallel
495x1000x90 mm	18mm plywood	85	3.30	perpendicular

gusset. The dial gauge came to rest on the other side of the reference bracket. Loading was applied through the arrangement as shown in figure 5.6 (without attaching the furnace). Loading was applied at a rate of 0.5 mm/min. in all these tests. The load level in kN was recorded manually. Displacements were sampled every three seconds and recorded every twenty seconds by the datalogging system.

In the case of steel gussets the failure of the joint commenced following the shearing off of some nail heads. In plywood gusseted joints the failure was due to yielding of timber or plywood fibres as the nail head pulled through.

5.6.3 TESTS ON STEEL GUSSETED JOINTS UNDER FIRE CONDITIONS

The main aim of these tests was to determine the load-slip characteristics of nails under fire conditions. The major variables in these tests were the types of protection used for the joint, the magnitude of the load and the direction of loading relative to grain direction.

Tests were conducted on all three protection types that were described earlier. The basis for the choice of loads is as follows: Under cold conditions (dead + snow load) nails are expected to carry a load of three times their basic nail loads (SANZ, 1987). Under fire conditions MP9 (SANZ, 1987) states that the joint (i.e. nails) must be capable of transferring at least dead load on the structure. Assuming the dead load is equal to the live load, this implies that, under fire conditions, the nails should carry an ultimate load of at least 1.5 times their basic nail load. In the tests however 1.8 times the basic nail load was chosen as the reference load, and this load was also used in most of the BRANZ verification tests . For 3.15 mm diameter nails NZS 3603: 1981 gives a basic nail load of 214 N. Therefore for the joint (6 nails) this load works out to be 1.8 x 214 x 6 = 2.31 kN. The lower loads were 75%, 50%, 25% of the highest load. i.e. 1.73 kN, 1.16 kN, 0.58 kN. In order to understand the load-deformation behaviour of the joint towards failure higher loads (3.47 kN and 4.63 kN) were also applied. When the highest load (4.63 kN) was applied it was decided to load the joint to failure at the end of the one hour test period. This was carried out as soon as the test period expired and this revealed the load-deformation behaviour of the joint at failure. Loading to failure took about one minute . The loads were applied both parallel and perpendicular to the face grain. The test arrangement was as shown in figure 5.6.

The performance of two layers of 14.5 mm gibboard protection was very good with the temperature on the glulam surface at the end of one hour reaching a maximum of only 106^{o}C. High temperatures on the glulam surface were observed when intumescent coating was used as protection. One layer

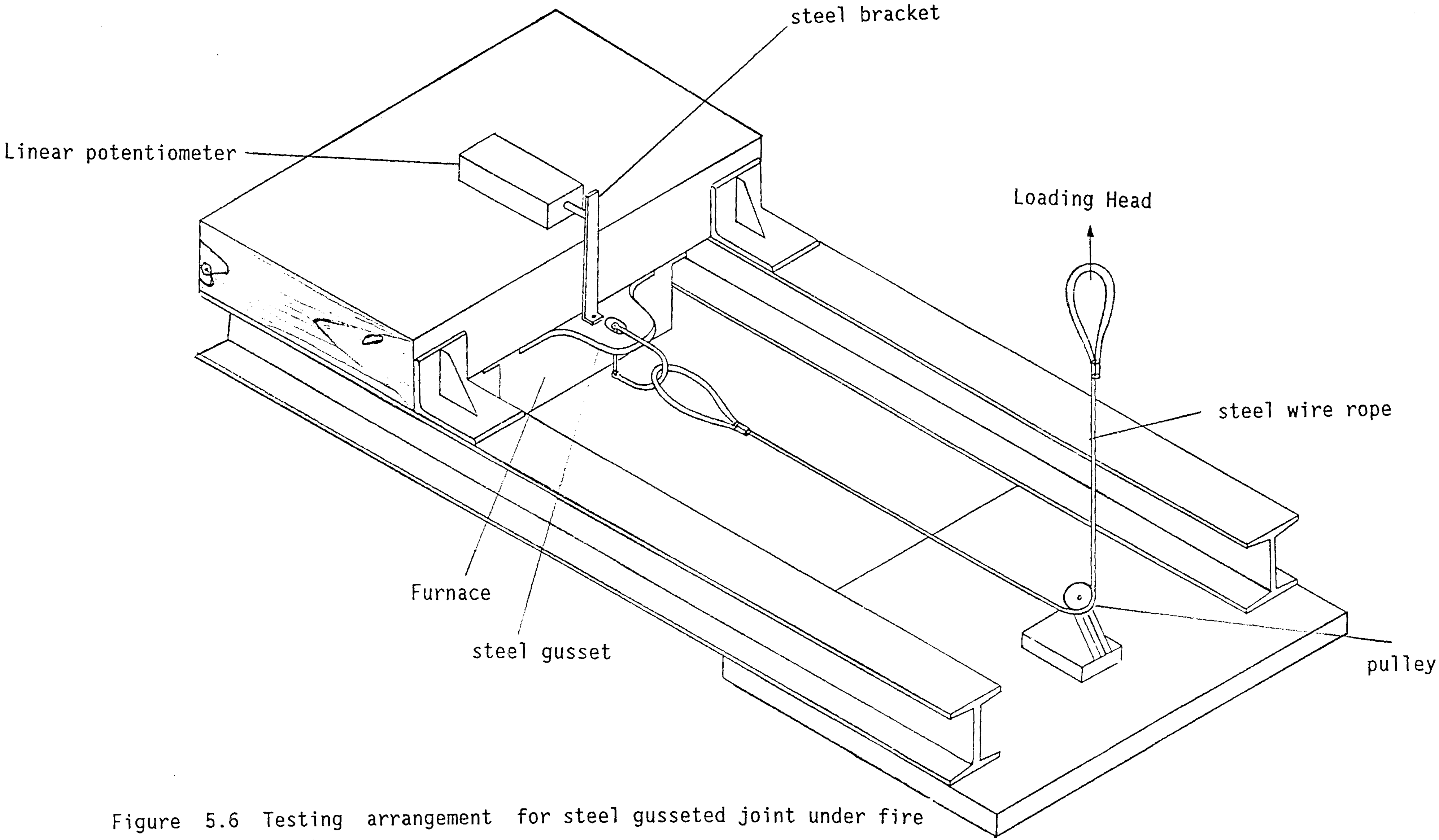


Figure 5.6 Testing arrangement for steel gusseted joint under fire conditions

of 19 mm gibboard gave reasonable protection.

For the gibboard protected joints the amount of charring increased as the thickness of protection being simulated decreased. Intumescent coating protected joints generally exhibited more charring than the gibboard protected joints as expected from the higher temperatures. These can be seen from figure 5.7. In tests where load is applied perpendicular to the face grain, cracks could be seen propagating along the nail lines. When nails of shorter length were used at a load of 2.31 kN, nail withdrawal at the far end of the gusset plate (from the loading point) occurred towards the end of the test.

5.6.4 TESTS ON PLYWOOD GUSSETED JOINTS UNDER FIRE CONDITIONS

These tests were conducted to determine the load-slip characteristics of nails in plywood gusseted joints under fire conditions. Gun nails of diameter 3.33 mm and length 85 mm were used, to attach the gusset to the timber specimen. The protection for the gusset was assumed to be 19 mm gibboard. The knowledge gained from tests on steel gusseted joints under fire conditions was very useful in deciding the loads for these tests. The reference load was still maintained to be 1.8 times the basic nail load, i.e. 2.31 kN. However at the lower end the joint was tested for only one load which is 50% of the reference load. At the upper end the joint was tested under both 3.47 kN and 4.63 kN as before. Loads were applied both parallel and perpendicular to the grain of the glulam. Thus a total of eight tests were carried out under fire conditions. The joint was loaded to failure immediately at the end of the test, when a load of 4.63 kN was applied.

In almost all of these tests, charring and cracking of the top layer of plywood gusset could be seen as shown in figure 5.8. Punching of the

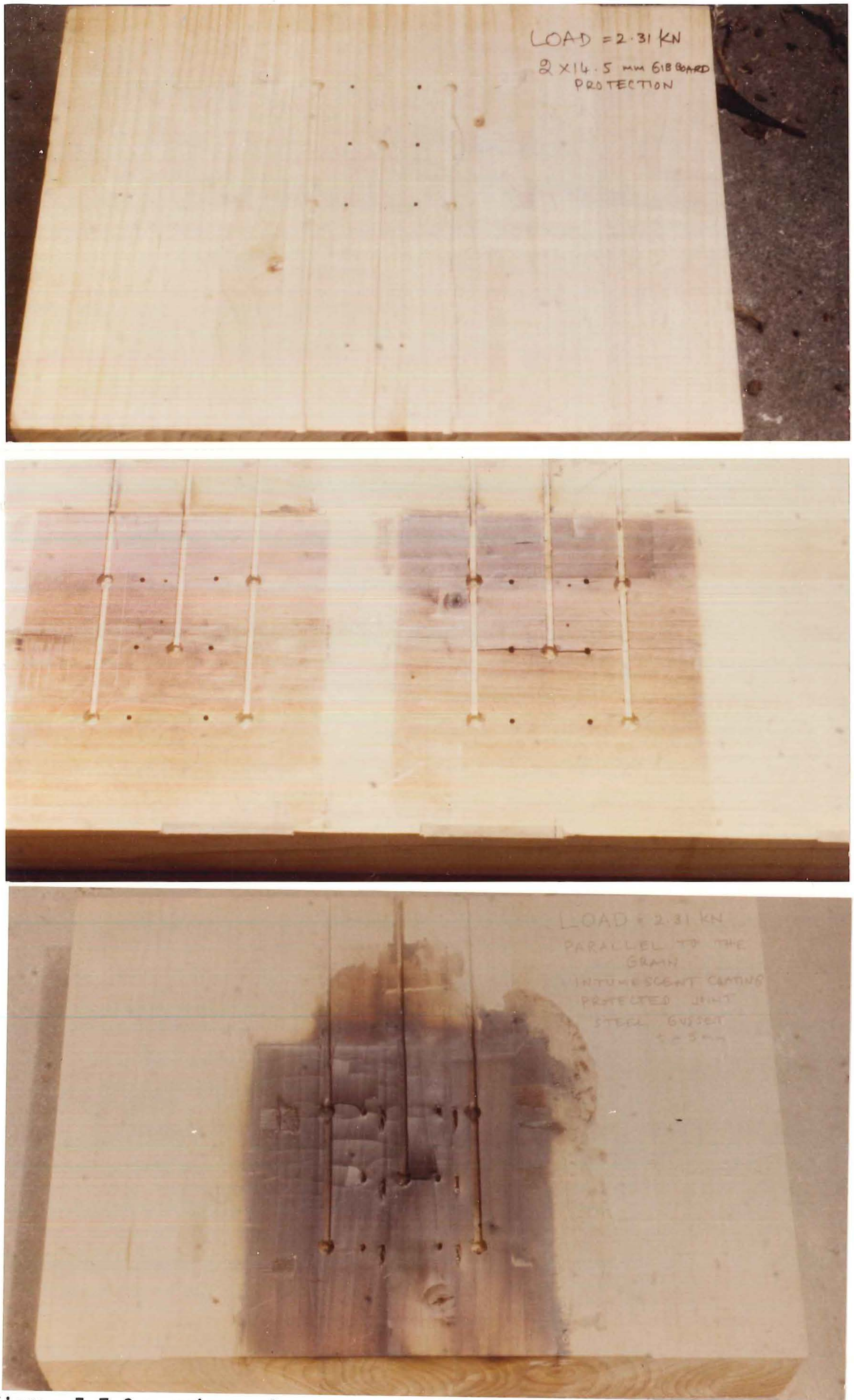


Figure 5.7 Comparison of amount of charring for various types of protection

nail heads into the plywood under applied loading, also occurred. However the nail shanks did not deform as much as they did in the case of steel gusseted joints.

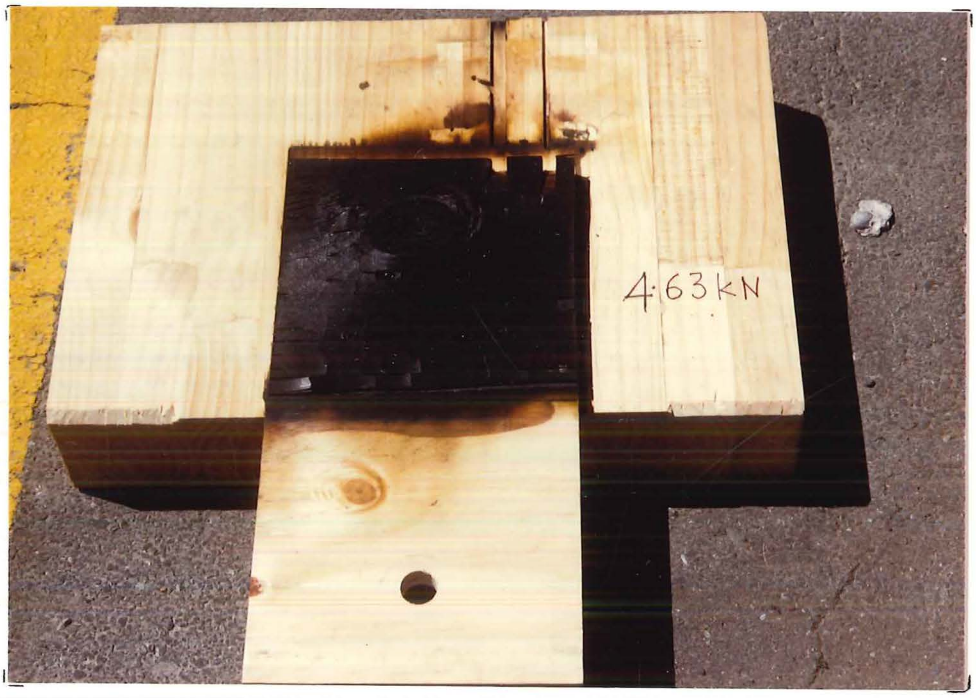


Figure 5.8 Plywood gusseted joint after 1 hr fire exposure

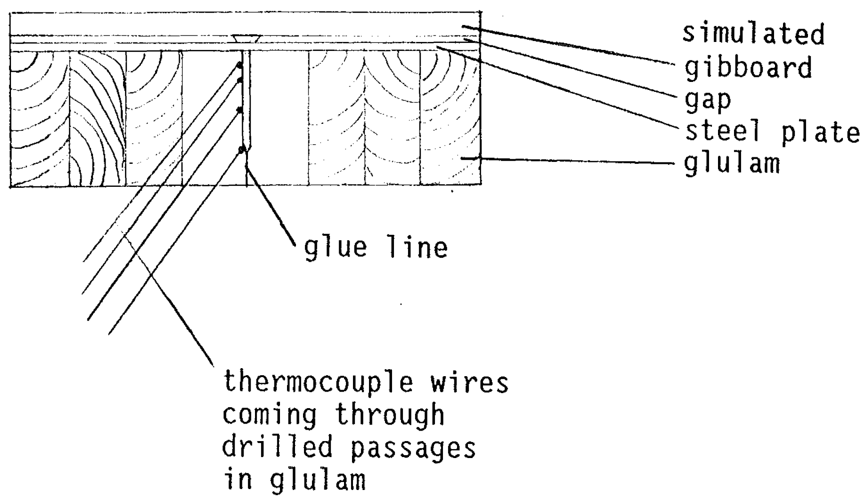
5.6.5 TESTS FOR DETERMINING THE NAIL TEMPERATURES

The aim of these tests was to determine the temperature development along the length of the embedded nail as the joint is exposed to fire. Nails in both steel and plywood gusseted joints were studied. The protection simulated in these tests was 19 mm gibboard. In the case of steel gussets the temperature development along the length of 75 mm long plain nails was monitored. In plywood gusseted joints the temperature development along one 85 mm long gun nail was recorded. In both cases the joint was unloaded.

As a first step in specimen preparation the nail was driven through the gusset plates into an unglued laminated joint in the glulam. The

Steel gusset

thermocouples at
10, 20, 40 and
75 mm from nail
head



Plywood gusset

thermocouples at
20, 40, 60 and
85 mm from
nail head

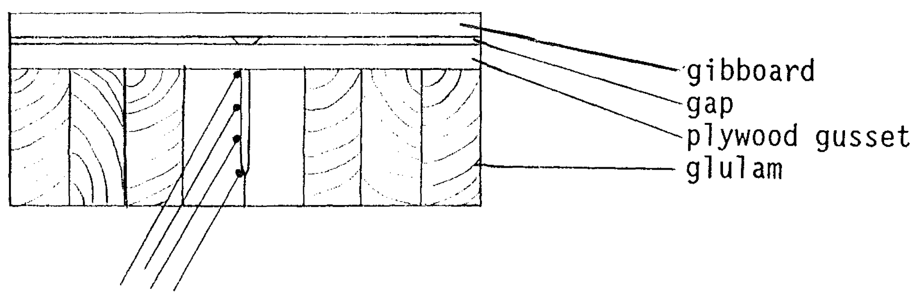


Figure 5.9 Nail temperature measurement - thermocouple locations

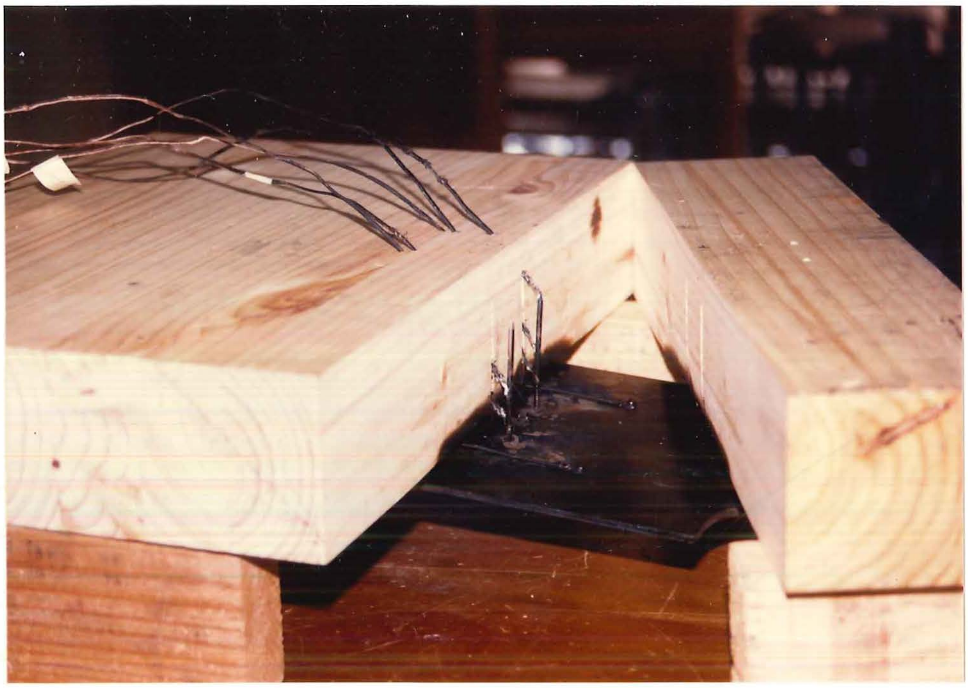


Figure 5.9 Nail temperature measurement - thermocouple locations

joint was then taken apart and the ends of thermocouples were welded to the nail shank and the thermocouple wires were routed through drilled passages in the timber specimen as indicated in figure 5.9. The nail was then positioned in the split timber specimen and the timber pieces glued back together. The remaining nails were driven in and the joint was allowed to set over night. The following day the joint was exposed to fire and the temperatures along the embedded nail shank were recorded.

5.6.6 TESTS TO DETERMINE THE BENDING STRENGTH OF NAILS

Tests were conducted to determine the bending strength of both gun nails and 75 mm long plain shank nails. The testing arrangement as shown in figure 5.10 was used. The span was 60 mm in all cases.

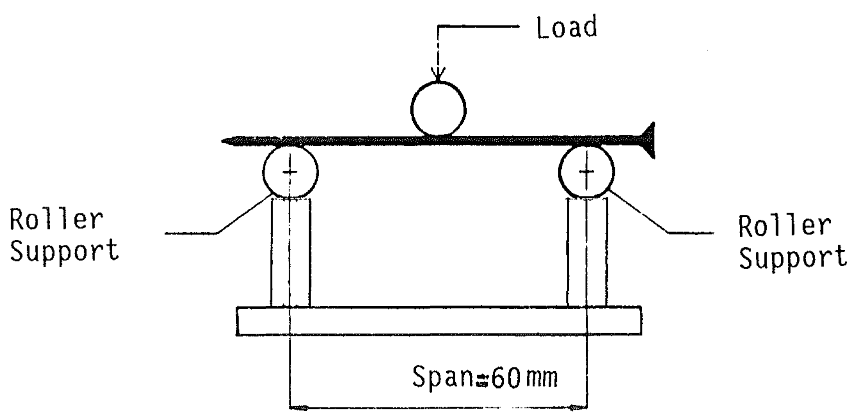


Figure 5.10 Simple bending test for nails

In the case of plain shank nails, the bending strength of nails of two different diameters was determined because the nails supplied consisted of a mixture of two different diameters. Three nails were tested in each category. Loading was applied in an Avery Universal testing machine and the displacement was measured by attaching the dial gauge to the loading head.

Initially the displacement was observed to be proportional to the loading. Once the maximum load was reached a plastic hinge formed, and the nail sustained constant load while undergoing more deformation. The test was discontinued when the displacements reached such a stage that the roller supports for the nail shank got pushed apart due to nail thrust.

CHAPTER SIX

RESULTS AND DISCUSSION

6.1 INTRODUCTION

The results of the experiments are discussed in this chapter. Numerous graphs and tables are used to substantiate the conclusions drawn. It is worth noting, that the results presented are from single tests only. There may be some scatter if the tests are repeated. However, they can reasonably be assumed to indicate overall trends.

6.2 LOAD-SLIP CHARACTERISTICS OF NAILS UNDER COLD CONDITIONS

The results of load-slip behaviour of nails in both steel and plywood gusseted joints are discussed. The major factors that are found to influence this behaviour are

(1) The manner of nail driving

(2) The length of the embedded nail shank

(3) The type of gusset used

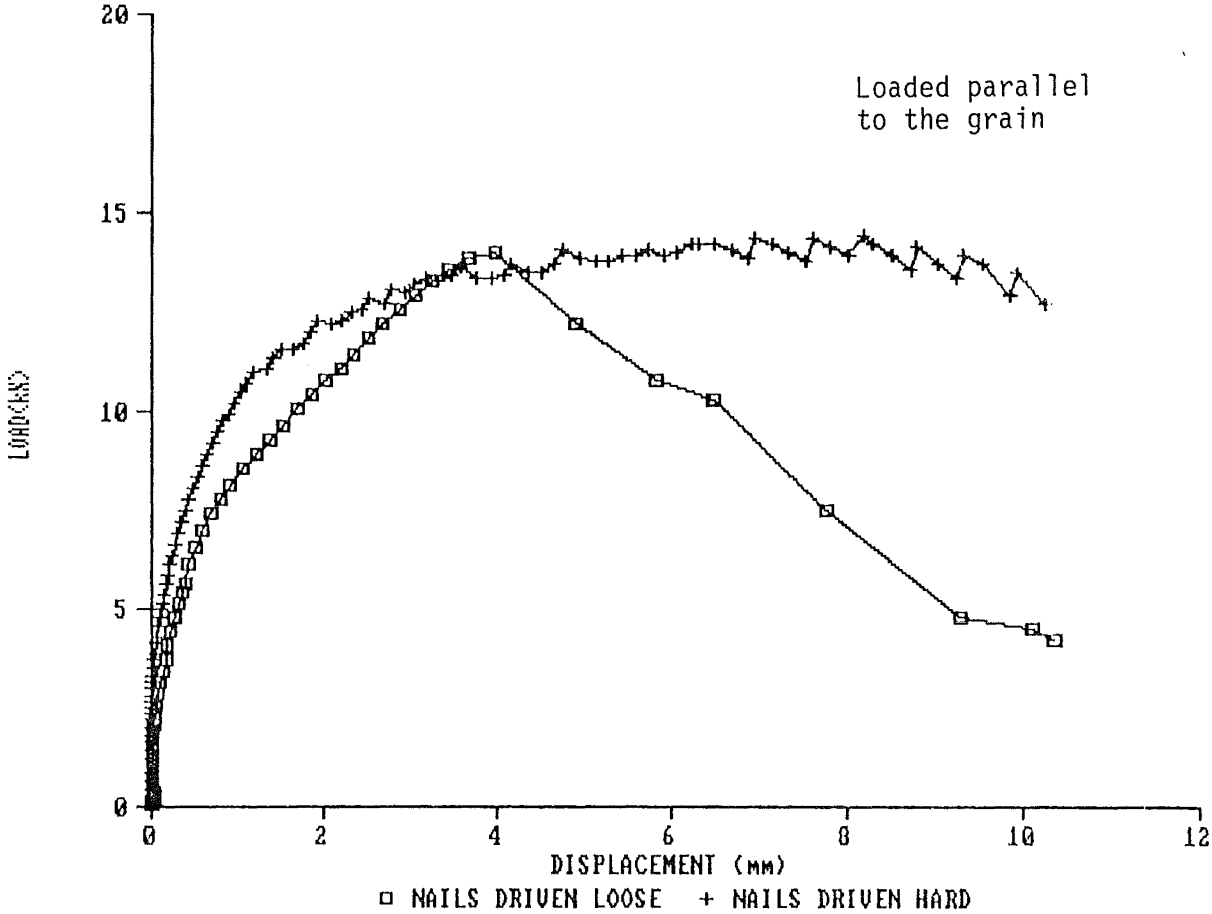
(4) The direction of loading relative to grain orientation

These factors are discussed individually in the paragraphs below.

STEEL GUSSETED JOINTS

6.2.1 EFFECT OF NAIL DRIVING

Tests were carried out with steel gussets only to assess the effect of nail driving on joint performance. The manner of nail driving did influence the load-deformation behaviour of the joint as shown in figure 6.1. In tests where the nails were driven harder, the joints were stiffer, and for loading perpendicular to the grain, sustained a higher load. In joints where nails were not driven hard, the drop in



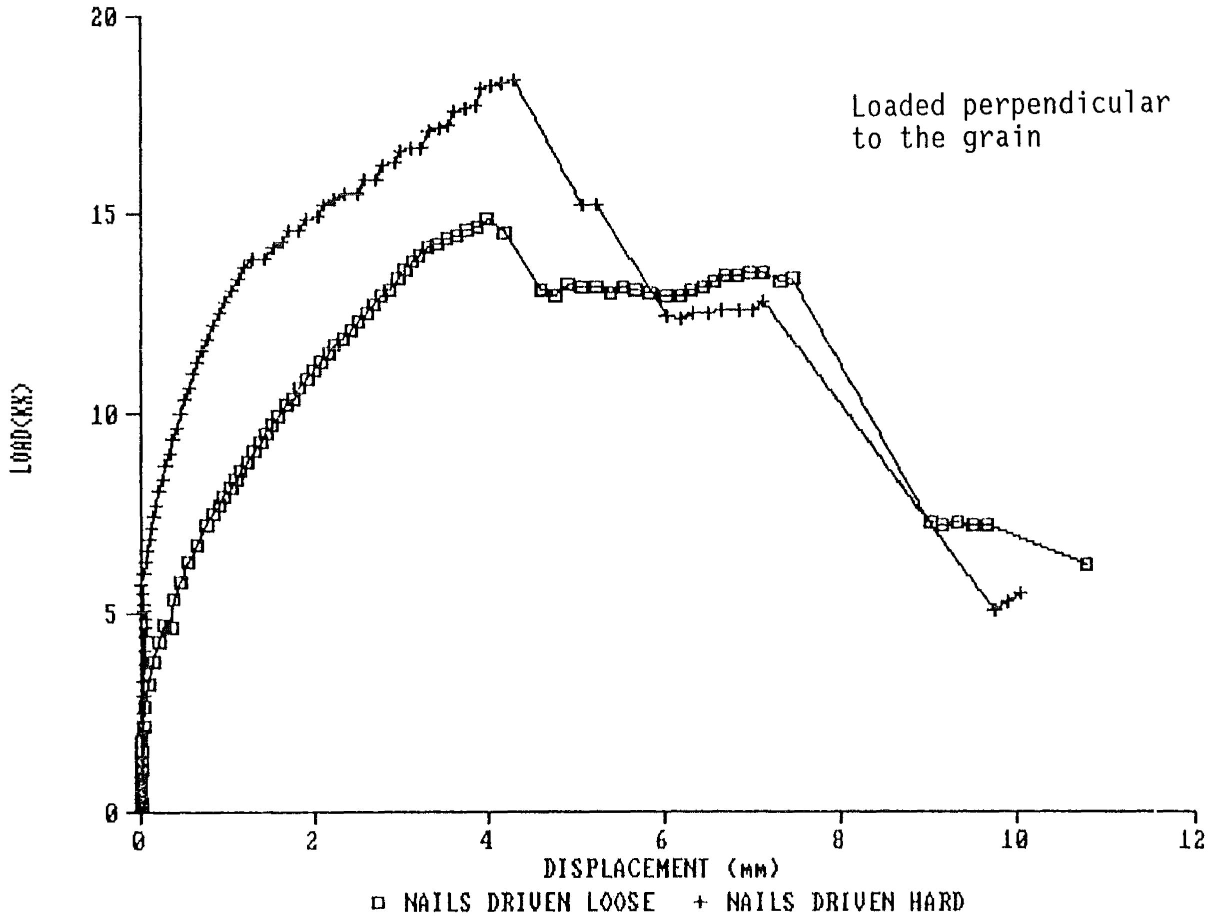


Figure 6.1 The effect of manner of nail driving on joint performance

load-carrying capacity tended to occur in steps, due to the shearing off of the nail heads. The improved load carrying capacity resulting from harder nail driving can not be relied upon in joint design as this might drop off due to relaxation of nails with time.

6.2.2 EFFECT OF LENGTH OF THE NAIL

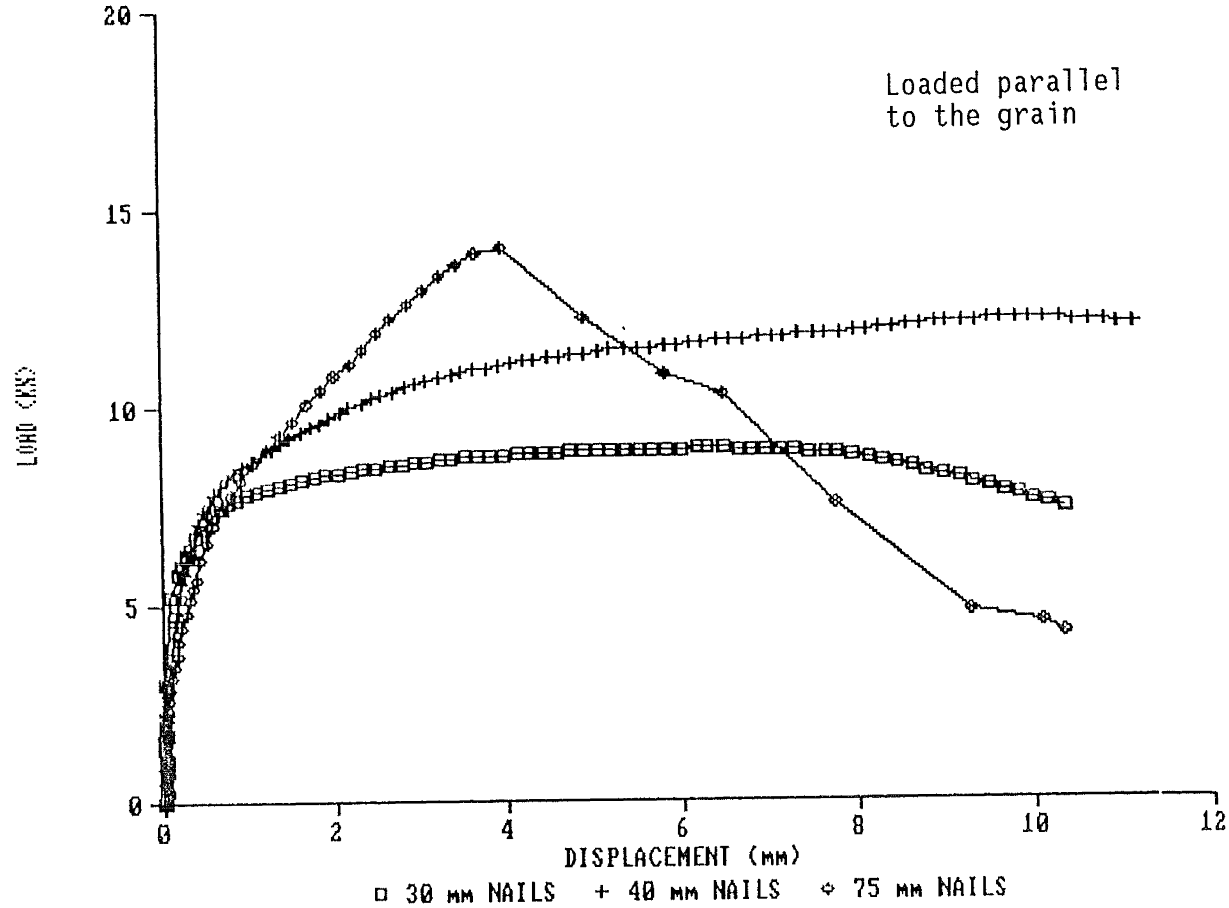
Figure 6.2 compares the effect of the length of the nail in steel gusseted joint performance. Three nail lengths were considered. At lower displacement levels of the joint (about 4 mm) the load carrying capacity appears to be proportional to the length of the nail. However longer nails (75 mm and 40 mm) exhibit a higher maximum load capacity compared to the shortest nail. The equal maximum load capacity of 40 mm and 75 mm nails appears to suggest that after a particular nail embedment, further increase in embedment may not influence the maximum load capacity of the joint.

The mechanism of failure was quite different depending on the length of the nail. With longer nails the nail heads sheared off at intervals causing a stepped failure. In the case of shorter nails the failure was due to nail shank withdrawal.

6.2.3 INFLUENCE OF LOADING DIRECTION RELATIVE TO GRAIN DIRECTION

The influence of loading direction relative to grain direction with respect to steel and plywood gusseted joints is discussed. In the case of steel gusseted joints this influence with respect to different nail lengths was also studied.

For 75 mm long plain nails the behaviour in both directions is quite similar as shown in figure 6.3. There is not much ductility due to nail heads shearing off.



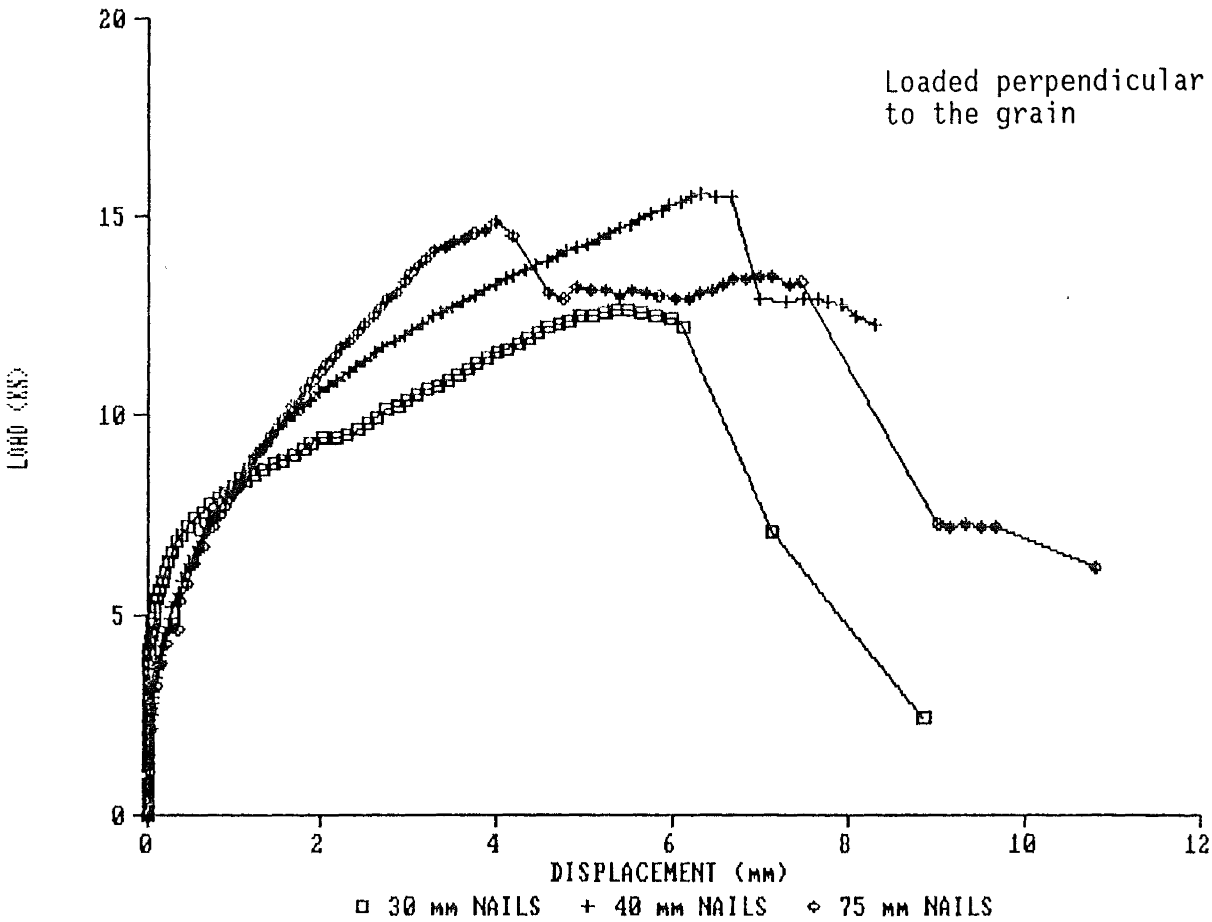


Figure 6.2 The effect of nail length in the load-slip behaviour of steel gusseted joint

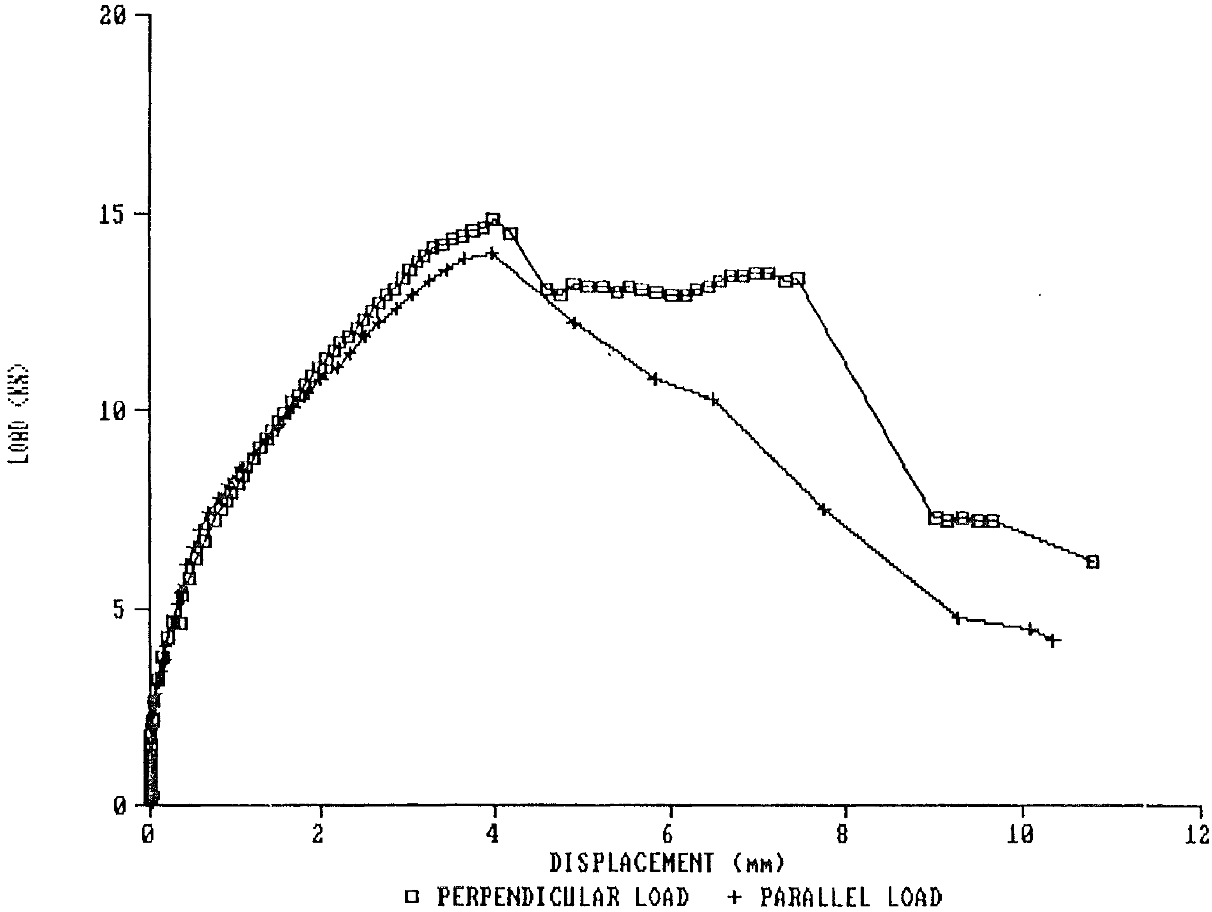


Figure 6.3 The influence of loading direction on joint performance 75 mm long nails, steel gusset

With 40 mm long galvanised nails a more distinct behaviour pattern can be observed as shown in figure 6.4. The load sustained is more when loaded perpendicular to the grain. However now the failure is more ductile, especially in the case of the joint loaded parallel to the grain. The behaviour of 30 mm long nails is similar to the 40 mm nails as indicated in figure 6.5, except that in this case the failure was quite sudden when loaded perpendicular to the grain.

PLYWOOD GUSSETED JOINTS

Only a single type of nail was used with the plywood gussets. The

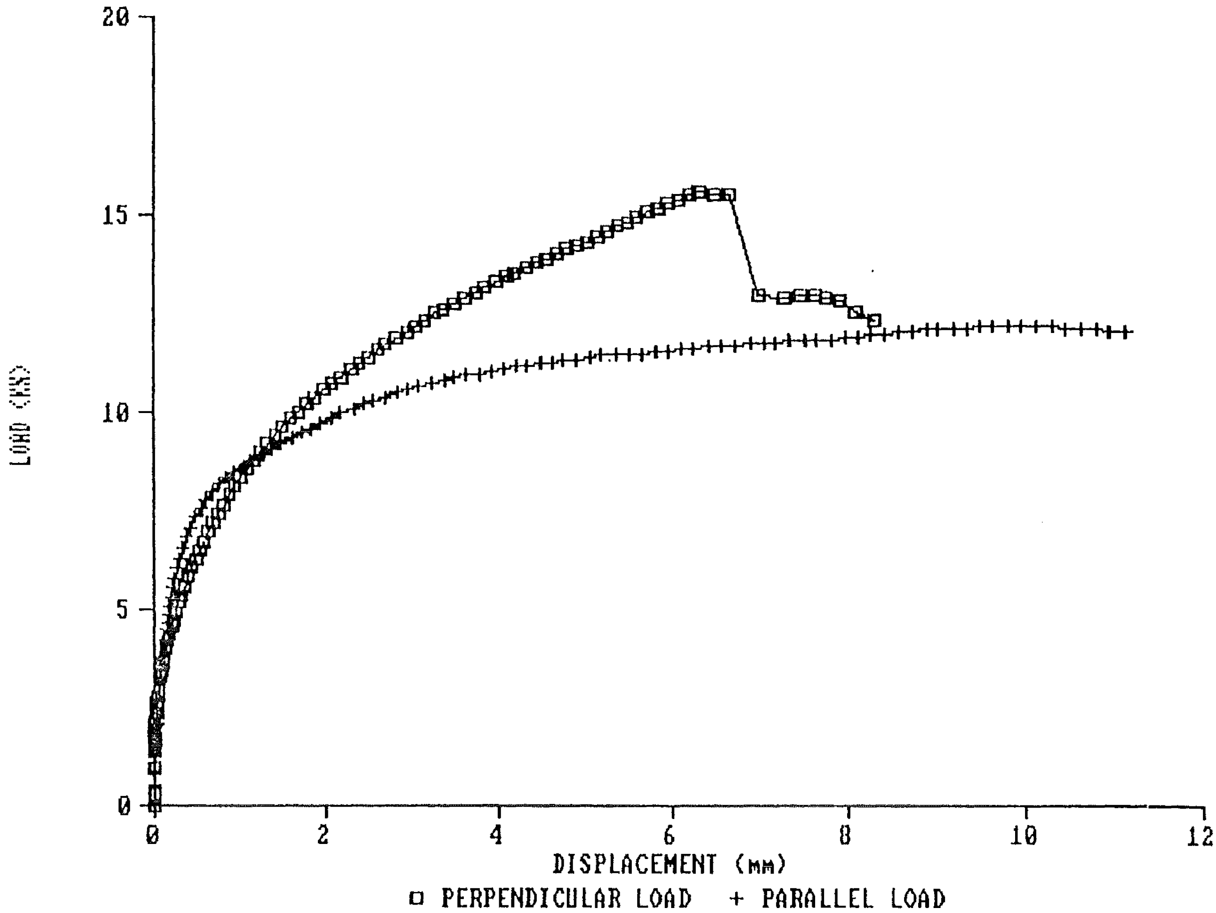


Figure 6.4 The influence of loading direction on joint performance
40 mm long nails, steel gusset

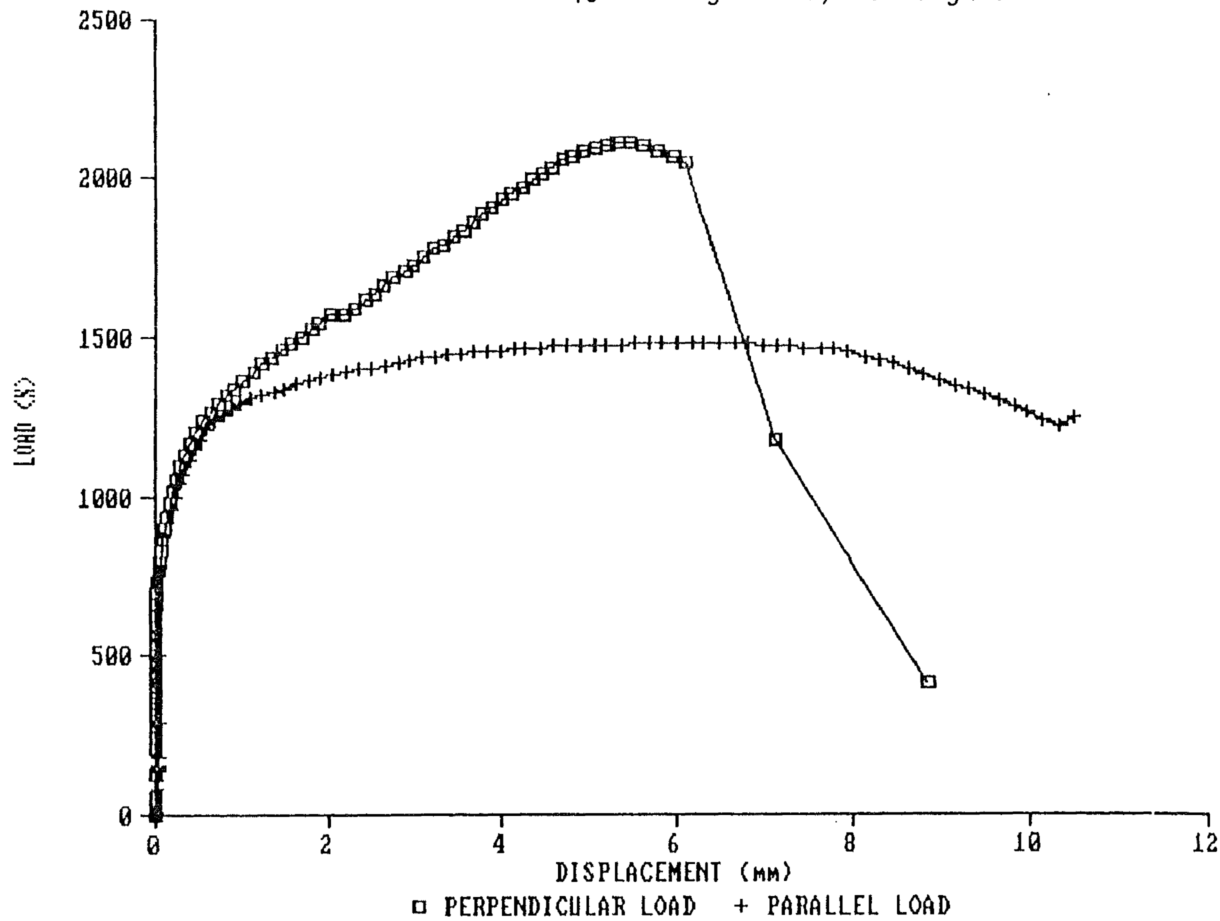


Figure 6.5 The influence of loading direction on joint performance
30 mm long nails, steel gusset

effect of direction of loading (relative to grain orientation) is shown in figure 6.6. The displacements are quite similar in both the grain directions.

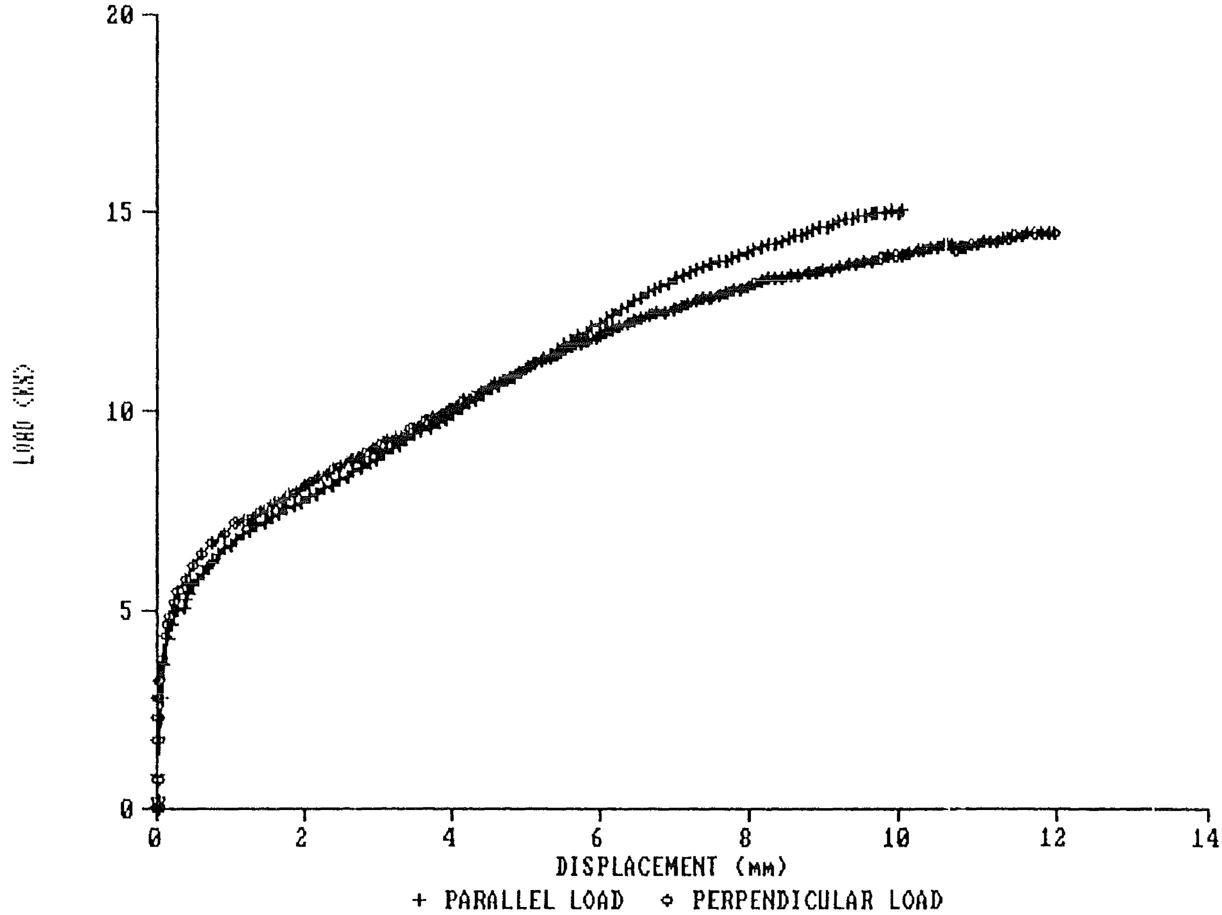
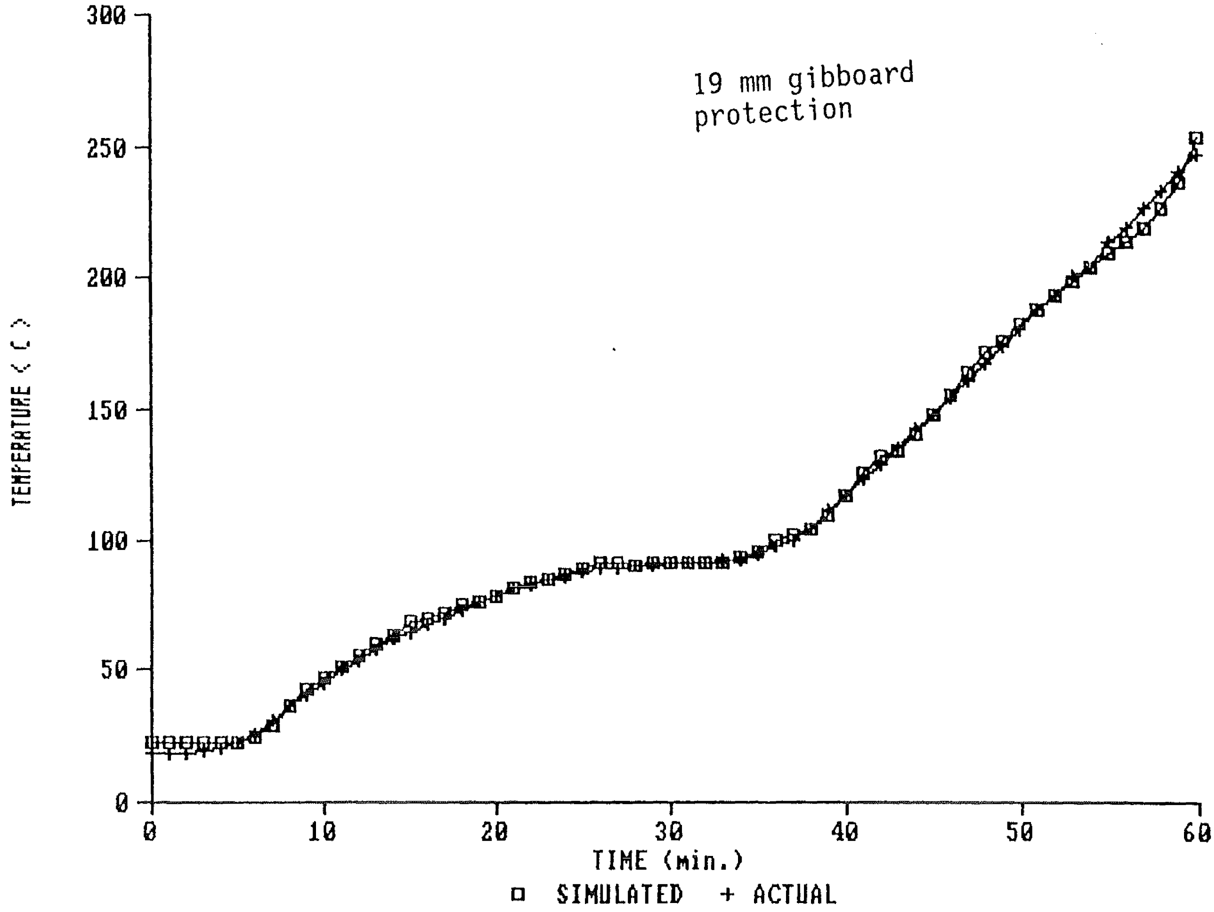


Figure 6.6 The influence of loading direction on joint performance
85 mm long nails, plywood gusset

6.3 LOAD-SLIP CHARACTERISTICS OF NAILS UNDER FIRE CONDITIONS FOR STEEL GUSSETED JOINTS

The influence of loads, the effectiveness of types of protection used and the role played by nails in joint performance are discussed. The simulated thermal exposure is compared against the actual temperature results of BRANZ study in figures 6.7a and 6.7b .



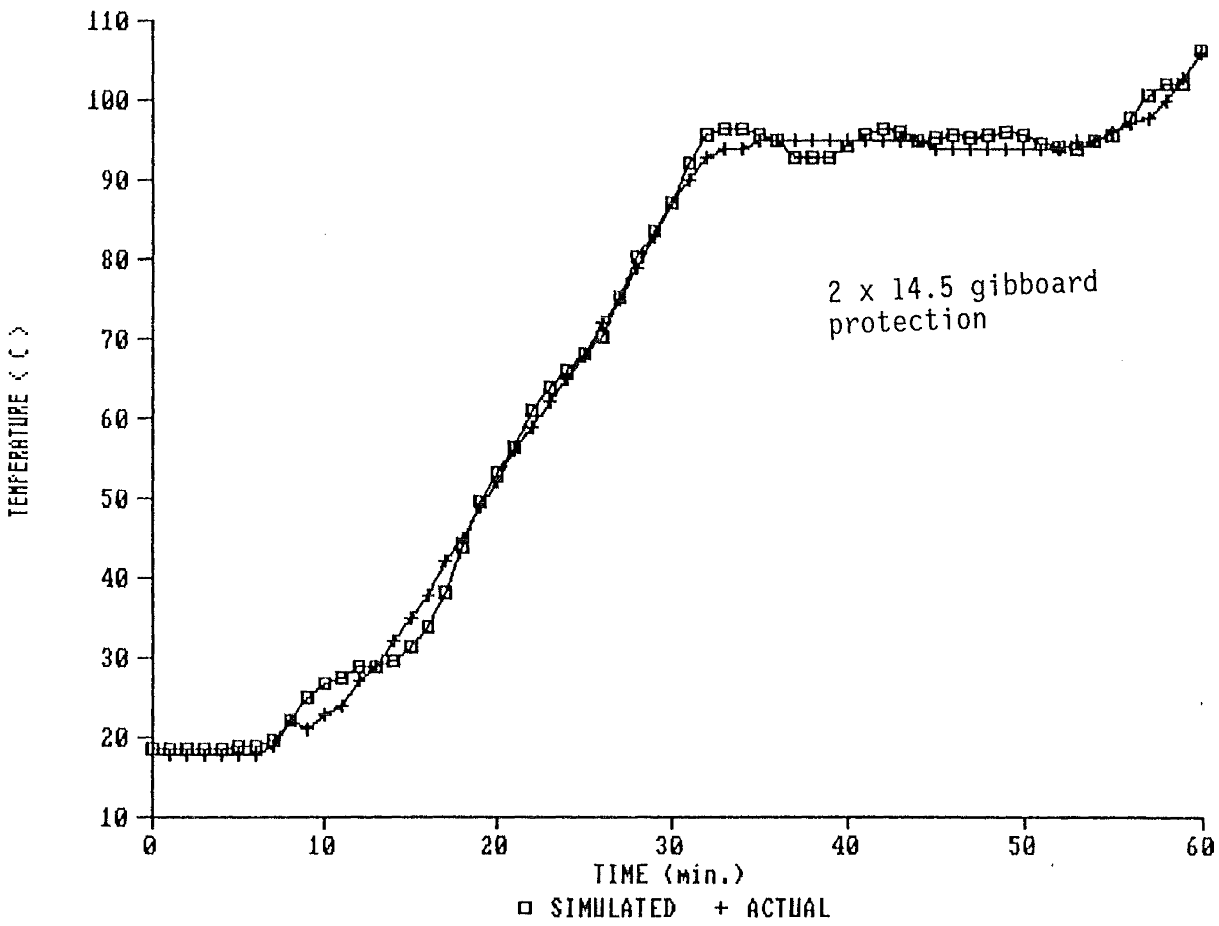


Figure 6.7 a Comparison of simulated and actual temperature results

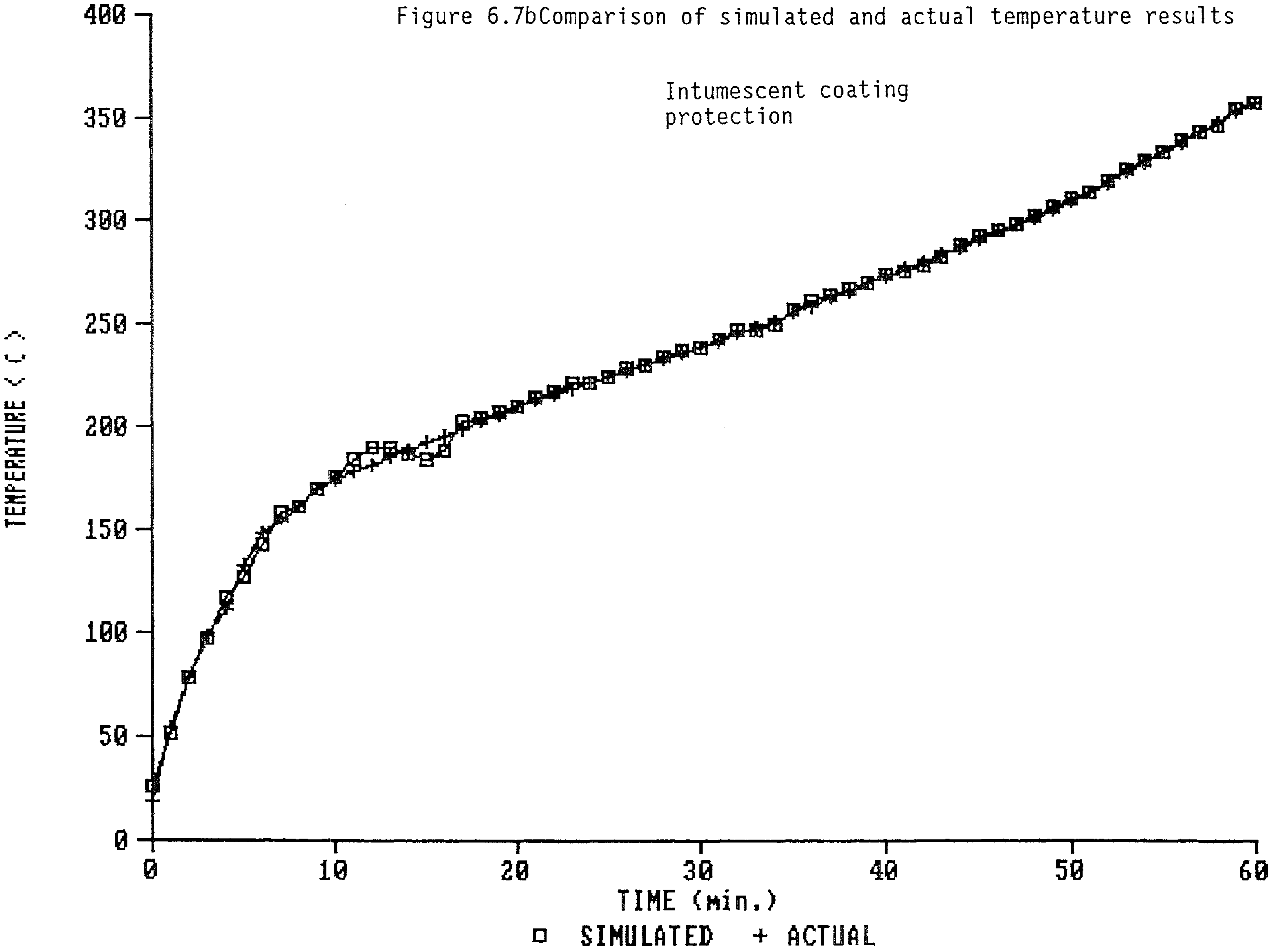


Figure 6.7bComparison of simulated and actual temperature results

6.3.1 COMPARISON OF EFFECT OF LOADS ON JOINT DISPLACEMENT

Figure 6.8 compares the displacements produced over 1 hour for two layers of 14.5 mm gibboard protected steel gusseted joint. When the loads were applied parallel to the grain note the higher initial displacements for higher loads (2.31 kN and 3.47 kN). In all load cases the displacements start to creep up after about 20 minutes. The temperature at this stage is about 50°C. Towards the end the displacements reach plateaus indicating the very good performance of the protection. A similar behaviour is observed when the loads were applied perpendicular to the grain.

Figure 6.9 shows the displacements obtained for a 19 mm gibboard protected steel gusset joint. Six different loads were applied. While for the highest load the displacement is distinctly higher, for lower loads there is a large amount of overlapping. The temperature at this stage is between 80°C and 120°C. The displacements exhibiting plateaus between 20 min. and 40 min. could be attributed to the phenomenon of moisture being driven off from the wood. Once this moisture is driven off, the displacements increase steadily and become proportional to the applied loads.

The few negative points indicated initially when the loads were applied perpendicular to the grain are due to slight distortion of the displacement measuring bracket. As before there was no significant initial displacements for the lower loads. The joint displacement under 2.31 kN appears to be greater than that under 3.47 kN for about half an hour. Towards the end the displacements become proportional to the applied loads. The difference in final displacements at higher loads is much greater than that at lower loads. This is distinctly visible in the

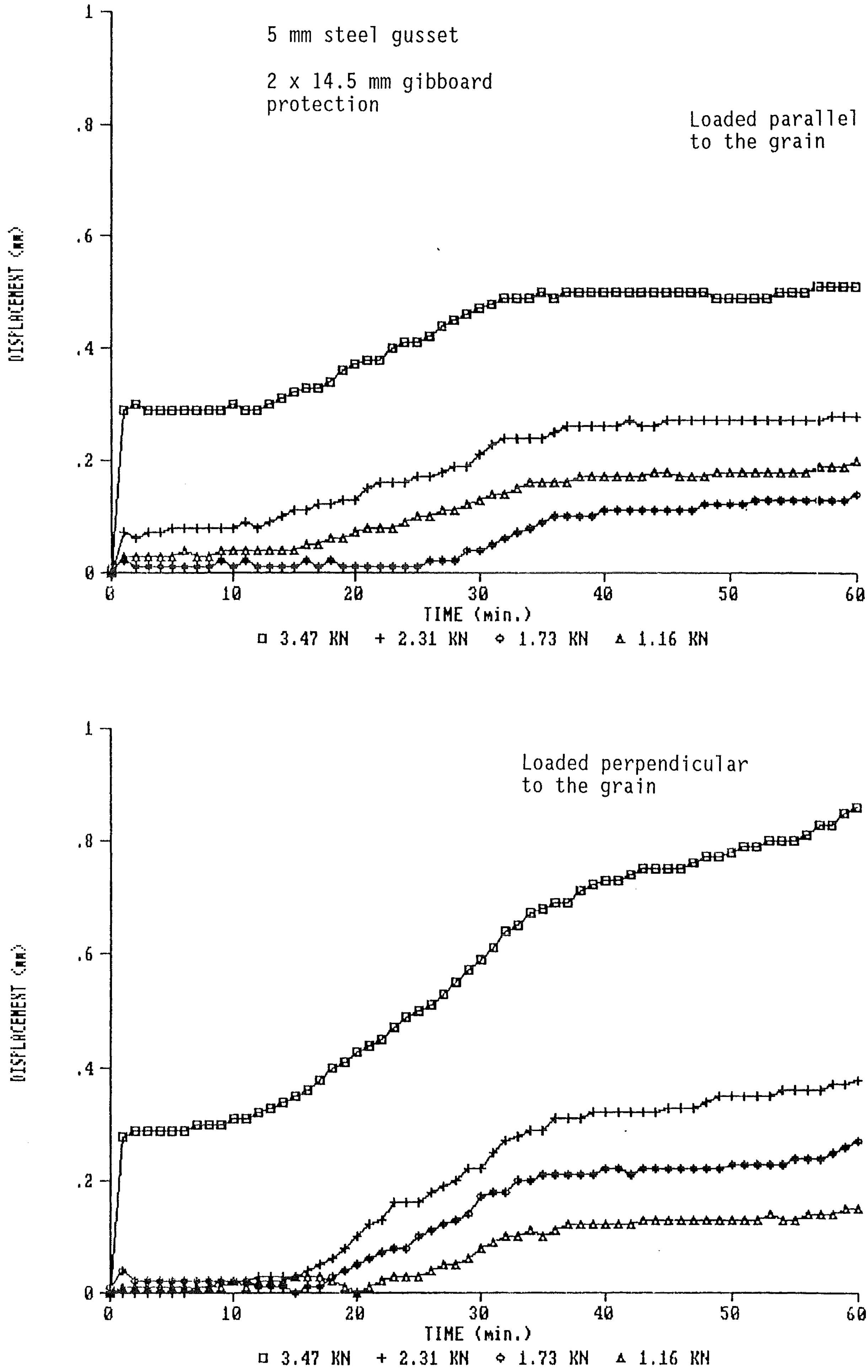
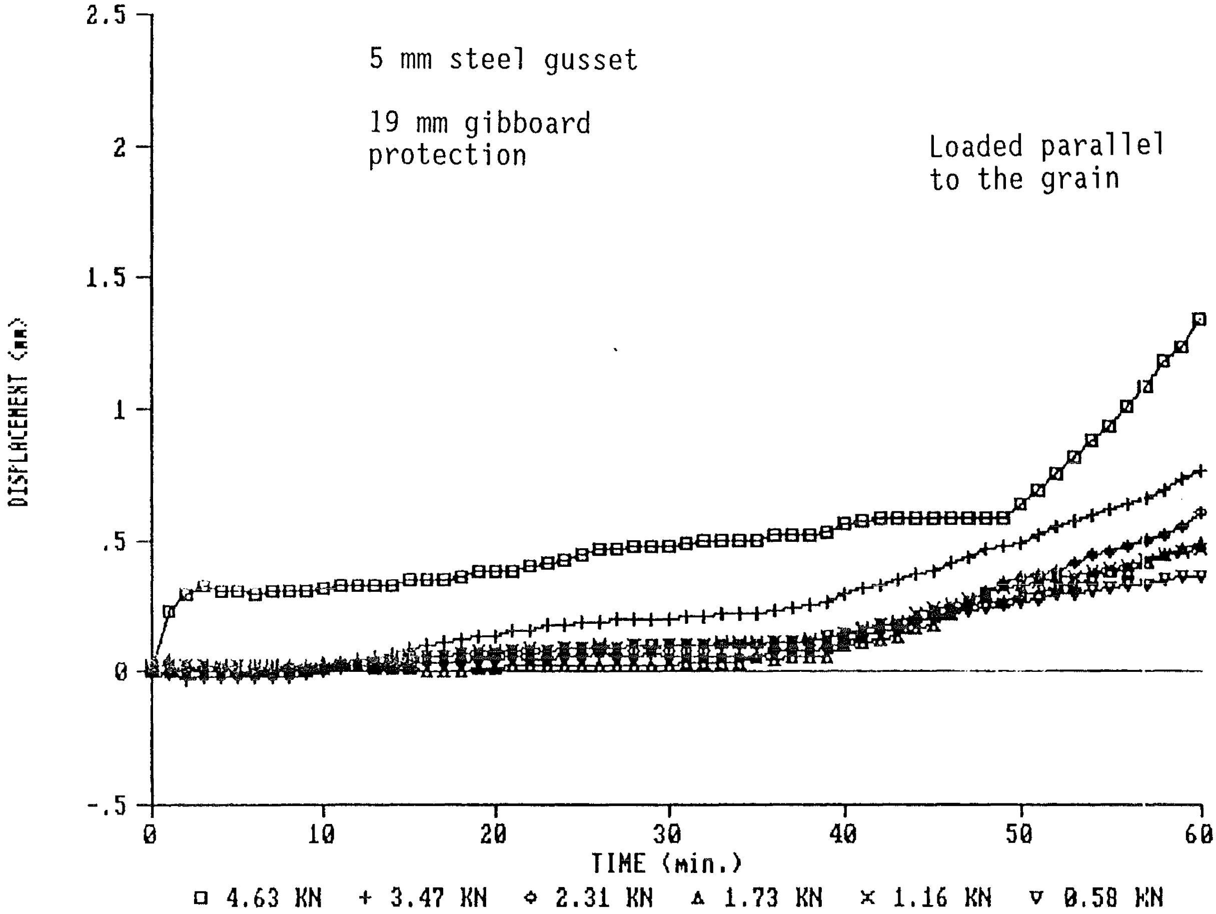


Figure 6.8 Effect of loads on joint displacement behaviour



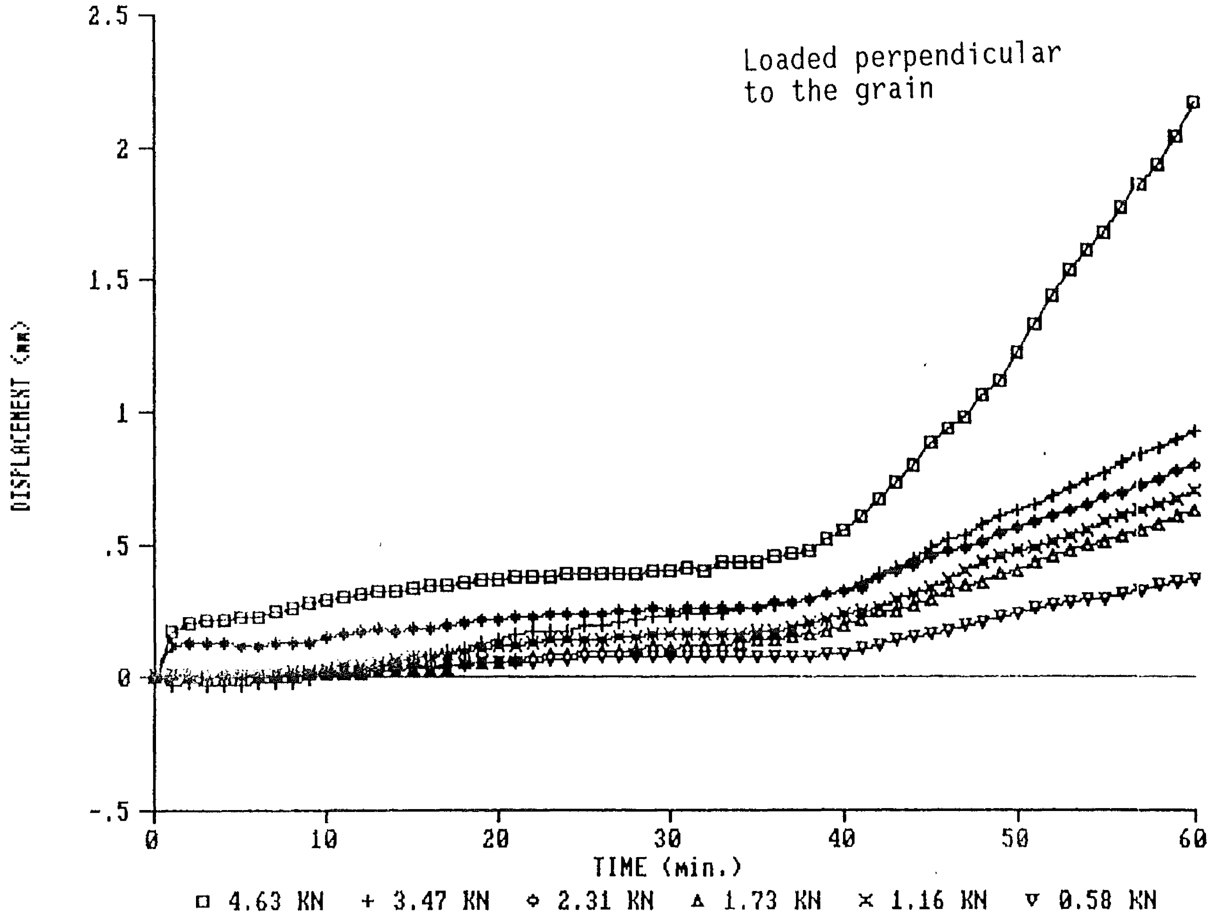


Figure 6.9 Effect of loads on joint displacement behaviour

case of the two highest loads used in these tests. This could be taken as one of the indications of the joint nearing its failure load.

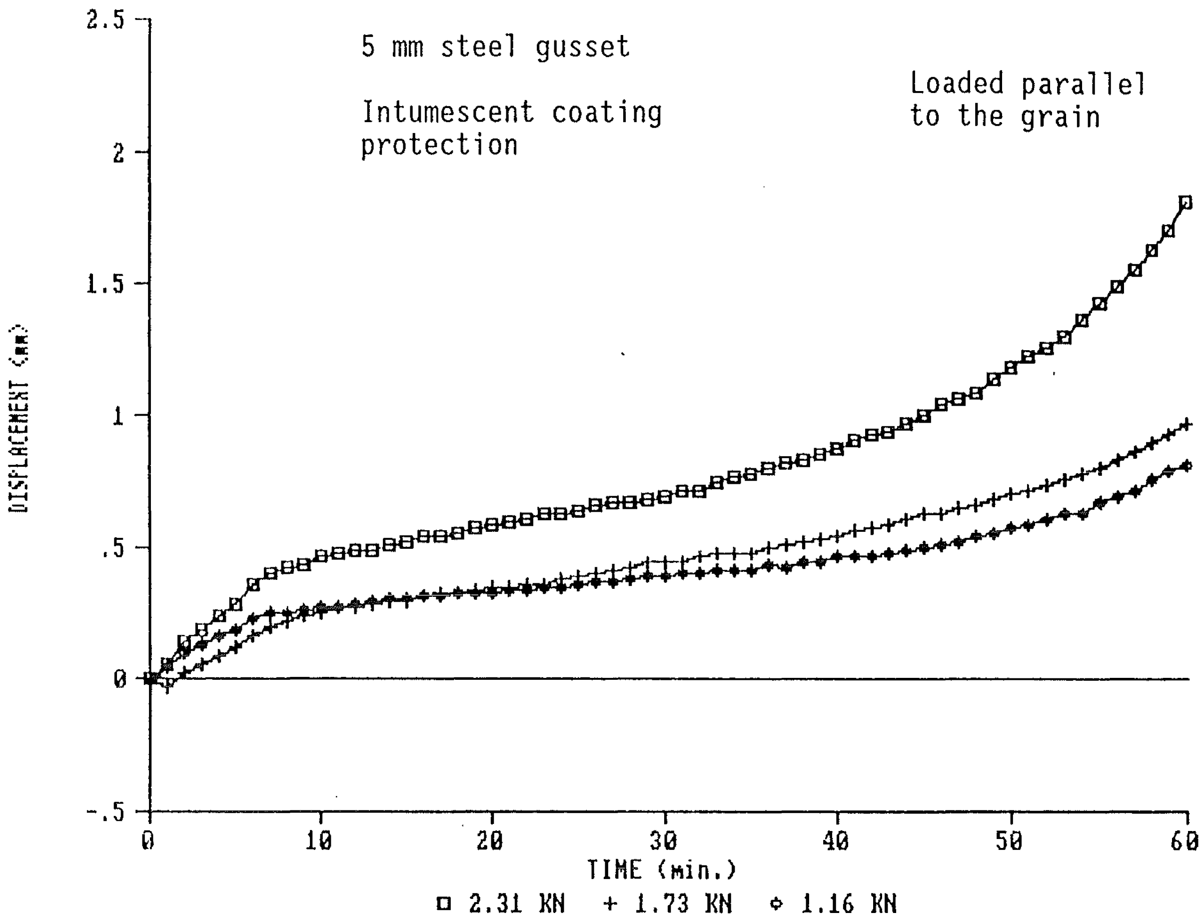
Figure 6.10 compares the displacements for intumescent coating protected steel gusseted joint for three different loads, applied both parallel and perpendicular to the grain. Here the loads do influence the displacement patterns. In the parallel direction the difference in 1 hour displacements between 2.31 kN and 1.73 kN is more than that between 1.73 kN and 1.16 kN.

When the loads were applied perpendicular to the grain, the displacements are proportional to the loads in the earlier stages. However, towards the end the two higher loads produce similar displacements and there is a marked difference between these displacements and that due to 1.16 kN.

6.3.2 COMPARISON OF EFFECTIVENESS OF PROTECTION

Figure 6.11 compares the displacement patterns corresponding to various protections when a load of 2.31 kN is applied. When loaded parallel to the grain the displacements for the intumescent coating protected joint are much higher than for the gibboard protection. However, for the gibboard protected joints, the thickness of protection does not seem to influence the displacement behaviour for most of the time. Only towards the end, the joint with lesser thickness of protection exhibits more displacement.

The joint undergoes more displacements in the perpendicular direction. While the displacements occurring in intumescent coating and two layers of 14.5 mm gibboard protected joints are at the two ends of the scale, the displacement due to 19 mm gibboard is between the two. If tested, the performance of a single layer of 14.5 mm gibboard protection



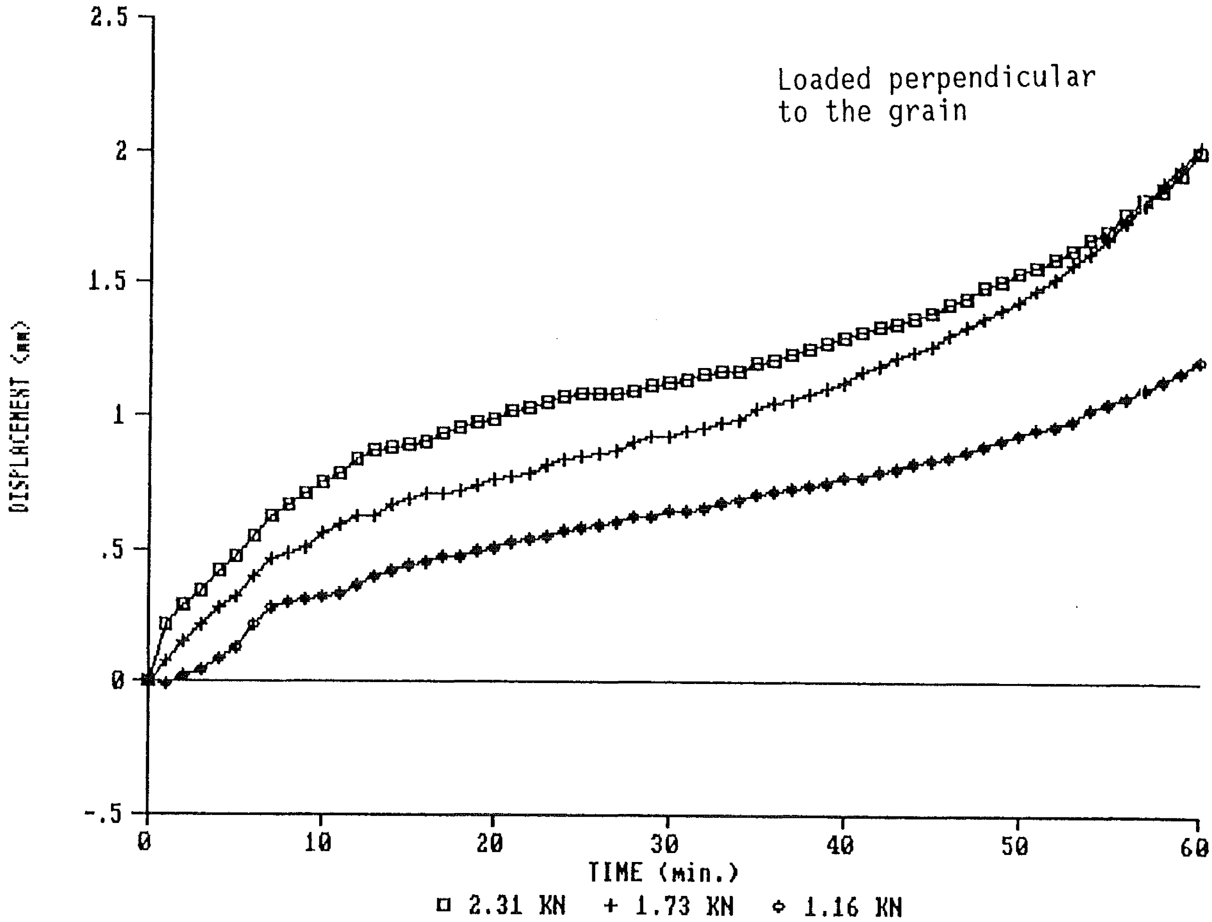
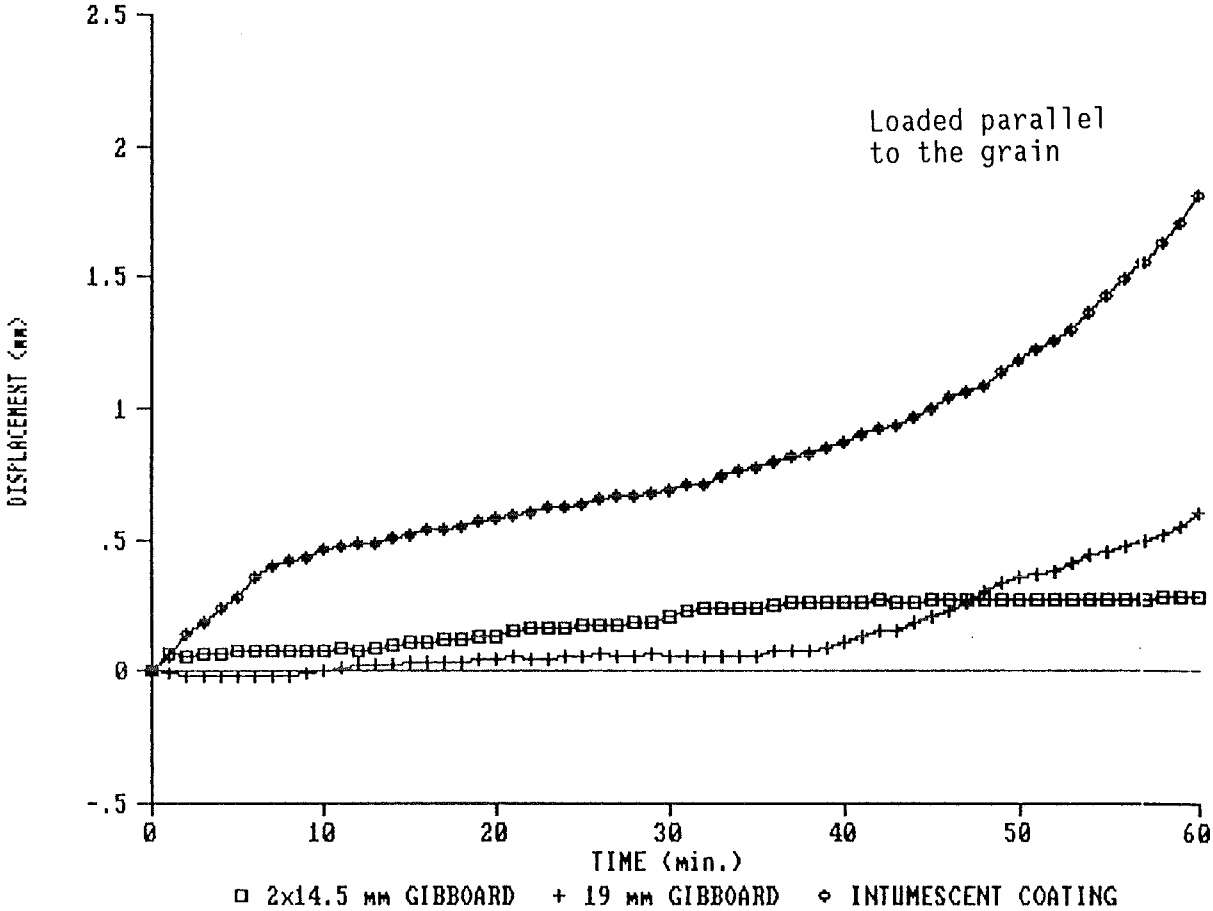


Figure 6.10 Effect of loads on joint displacement behaviour



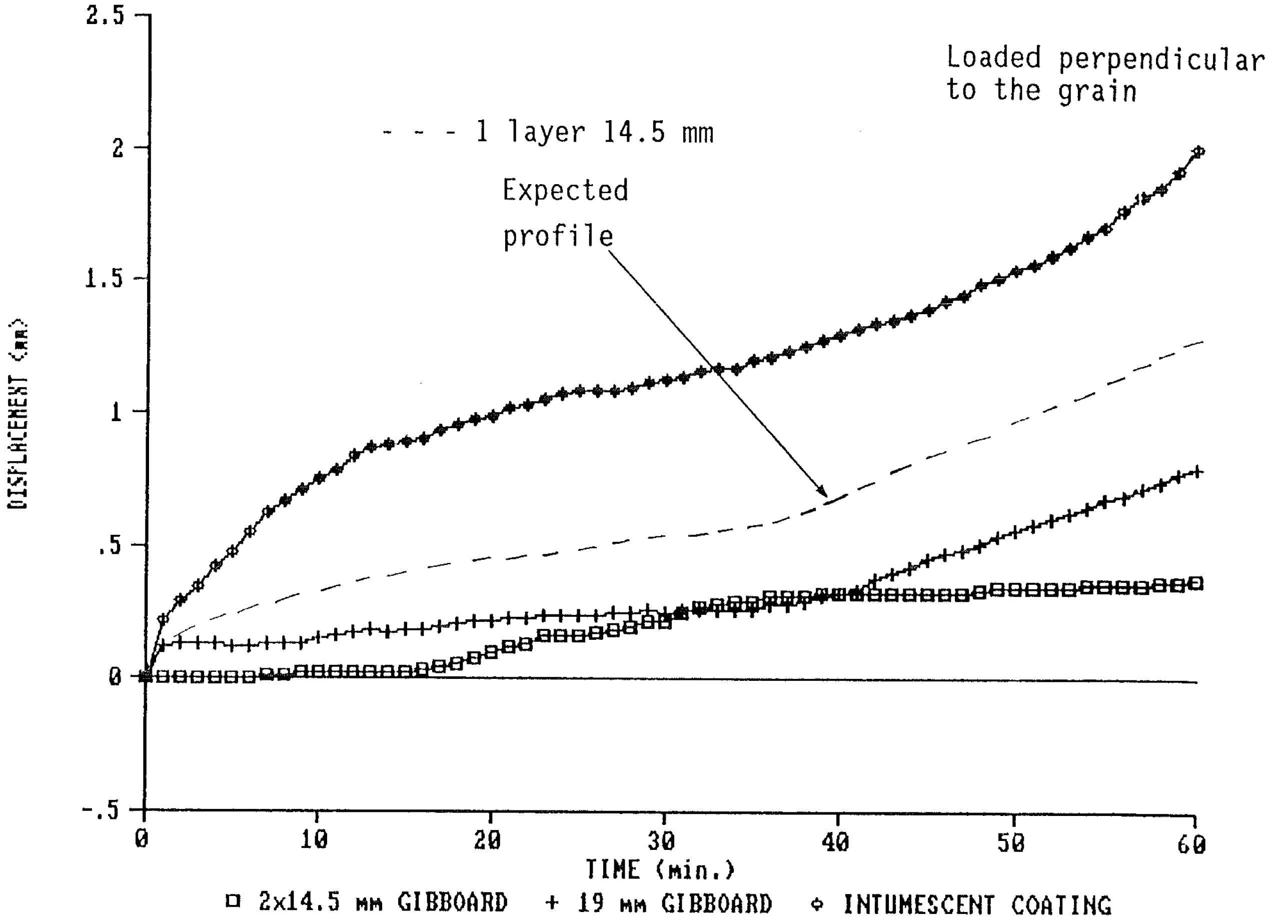


Figure 6.11 Comparison of effectiveness of protection on joint performance (2.31 kN)

is expected to lie between intumescent coating and 19 mm gibboard protection as shown in dotted lines in figure 6.11.

Figure 6.12 shows the displacement results for a load of 1.73 kN. In the parallel direction the displacement values are much reduced compared to 2.31 kN. When the load is applied perpendicular to the grain the displacements resulting from the intumescent coating protected joint are much higher than those corresponding to gibboard protections .

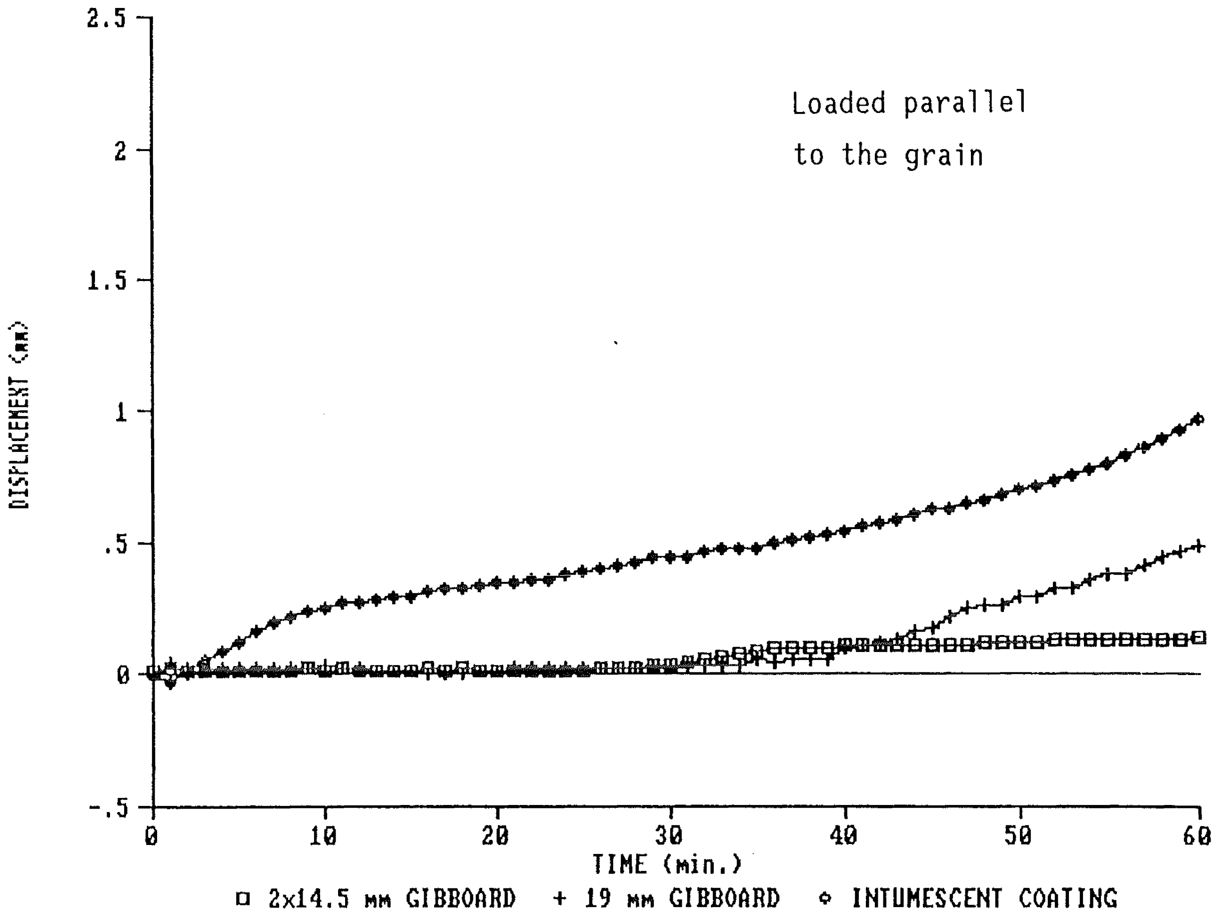
Figure 6.13 compares the displacement results for a load of 1.16 kN in two different grain directions. The displacements are much reduced due to the reduced load level.

6.3.3 COMPARISON OF DISPLACEMENTS IN DIFFERENT GRAIN DIRECTIONS

Figure 6.14 compares the displacements in different grain directions for a 19 mm gibboard protected joint, under a load of 2.31 kN. The displacement in the perpendicular direction is greater than that in the parallel direction. Similar behaviour was observed for other loads.

Figure 6.15 compares the displacements due to a load of 2.31 kN for a joint protected with two layers of 14.5 mm gibboard. Again the displacements are higher in the perpendicular direction. Similar behaviour was observed for this protection under the action of other loads.

Figure 6.16 indicates the displacements for intumescent coating protected joint for a load of 2.31 kN. The joint undergoes more deformation in the perpendicular direction than in the parallel direction.



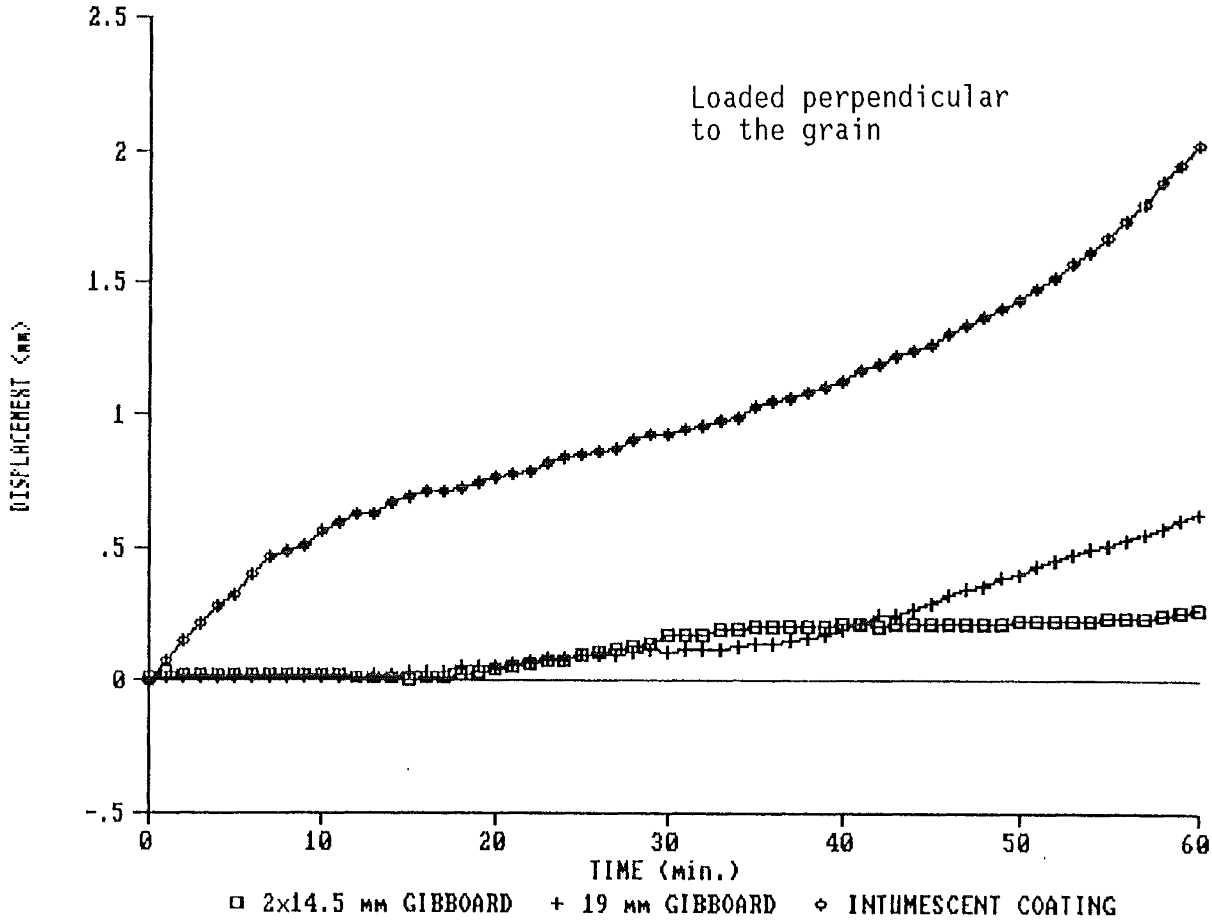
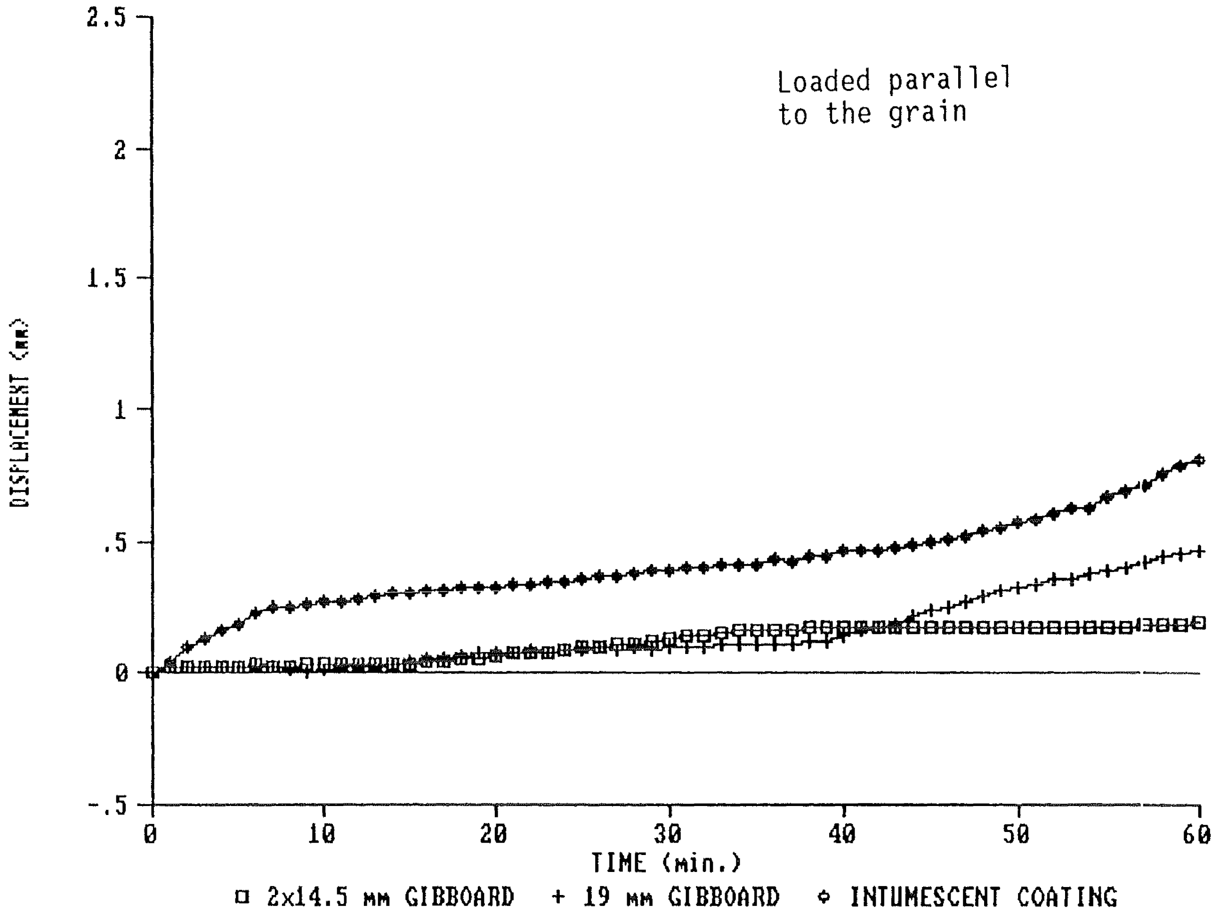


Figure 6.12 Comparison of effectiveness of protection on joint performance (1.73 kN)



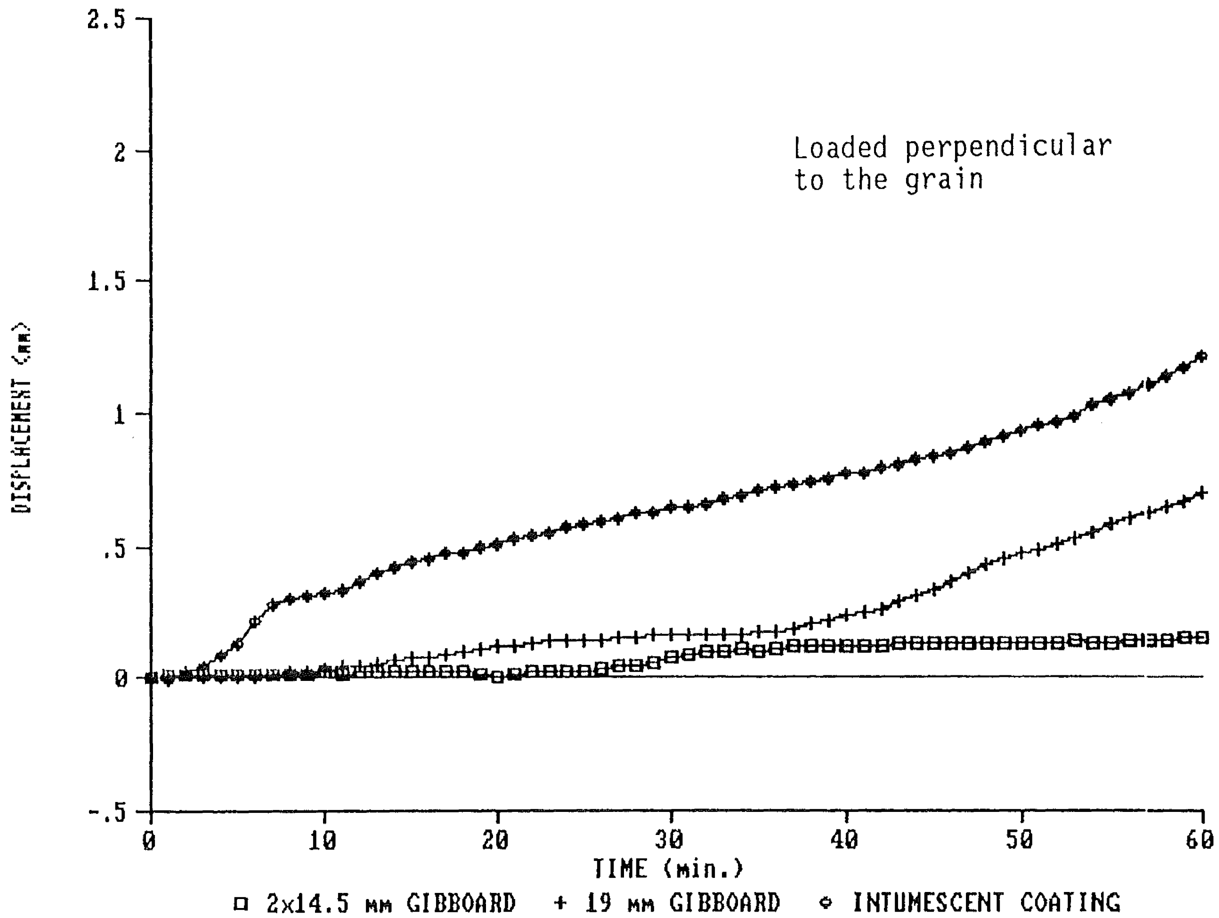


Figure 6.13 Comparison of effectiveness of protection on joint performance (1.16 kN)

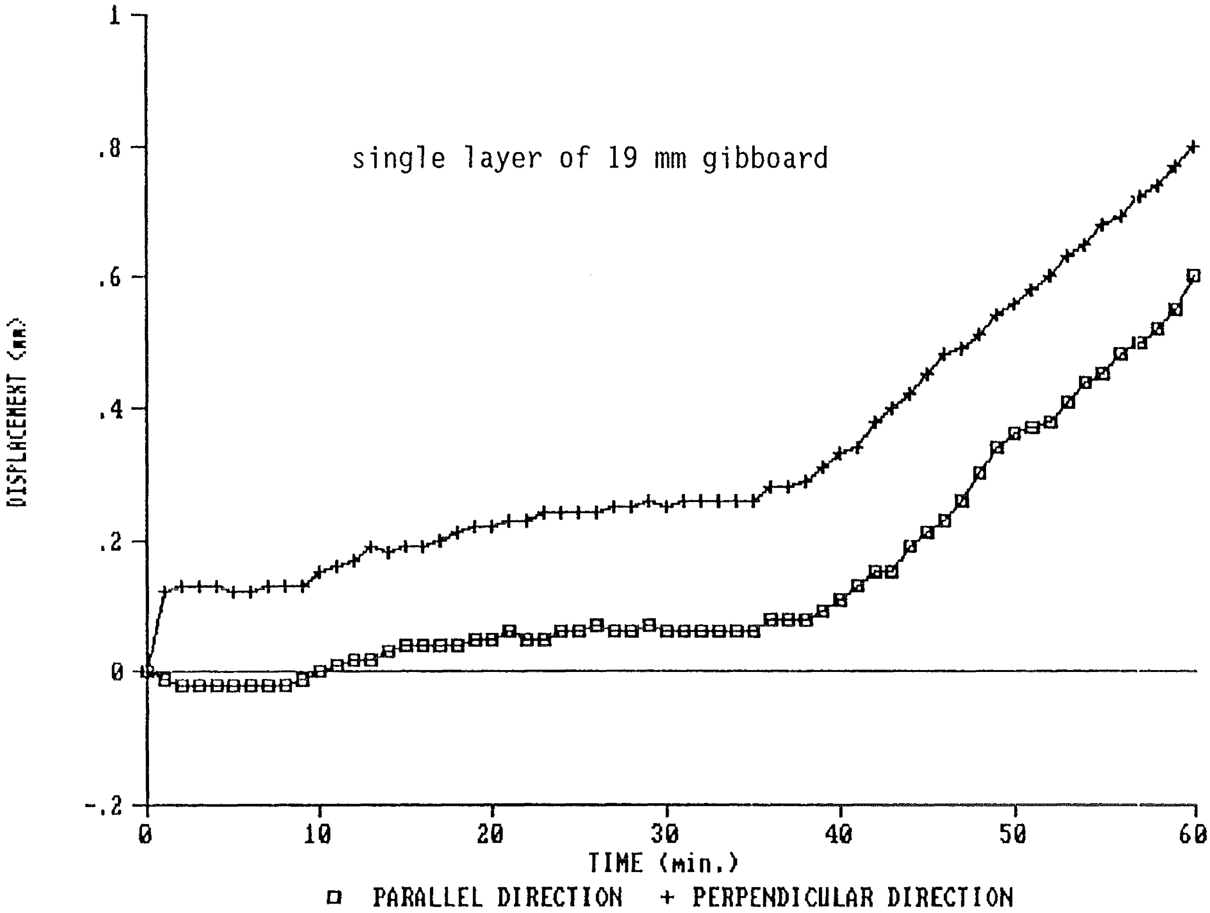


Figure 6.14 Comparison of displacement in different grain directions

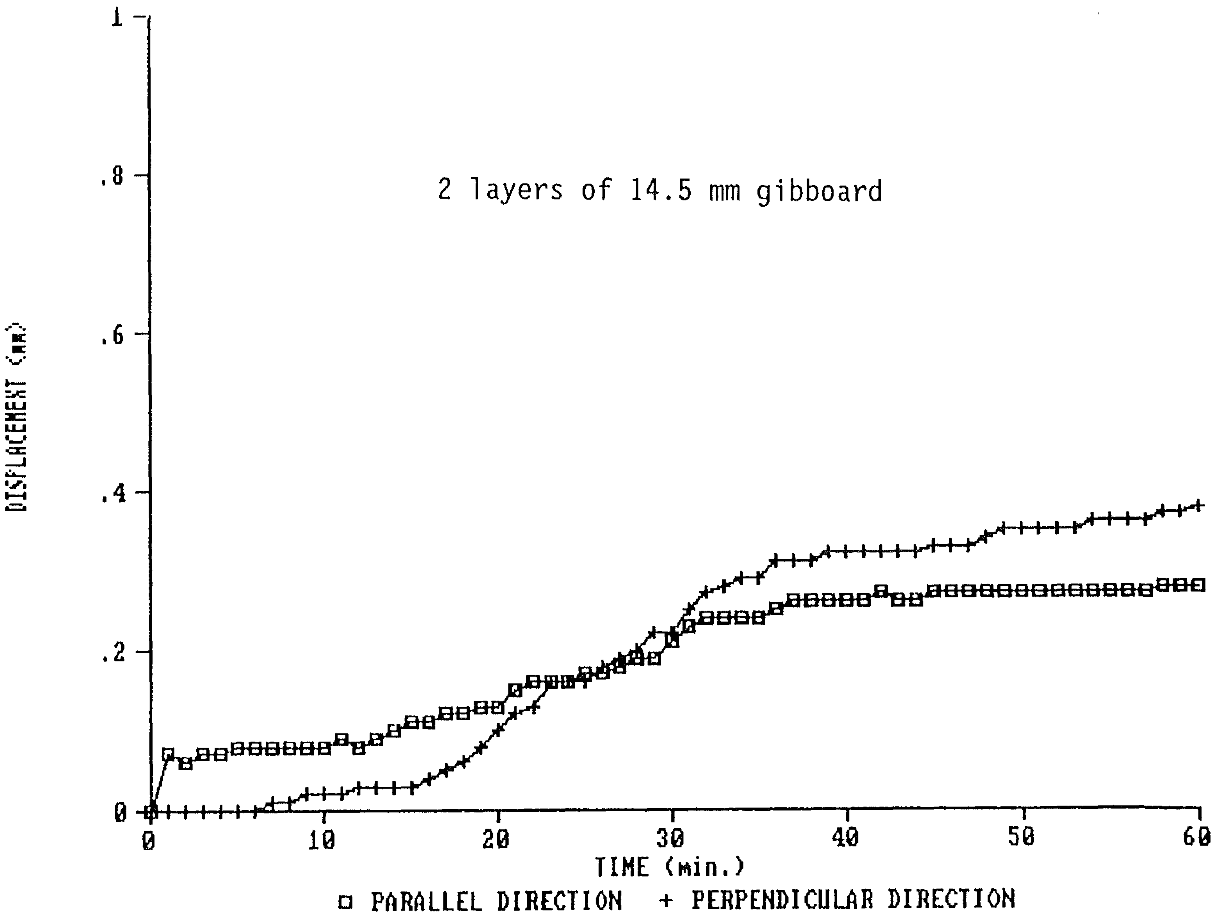
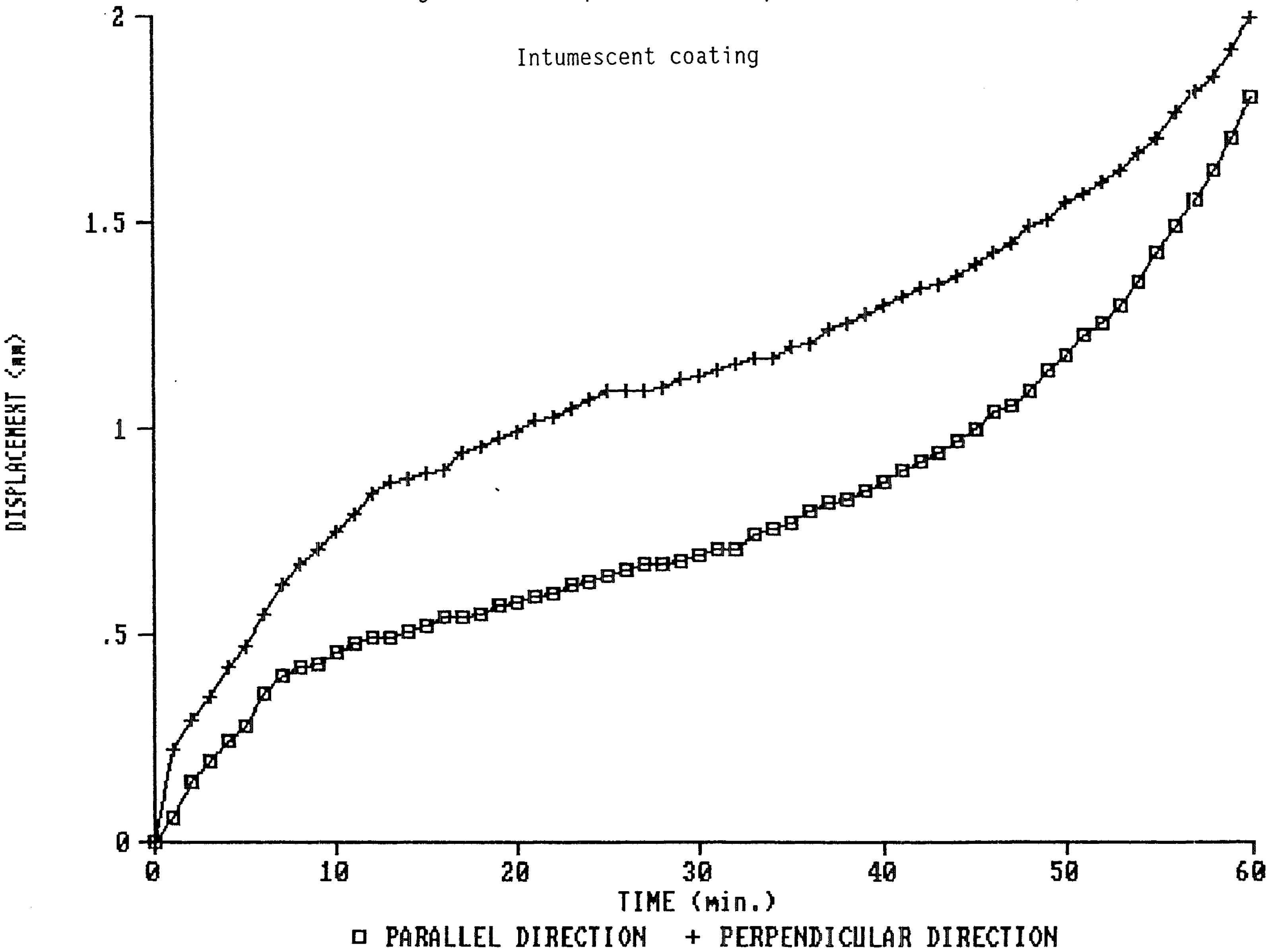


Figure 6.15 Comparison of displacement in different grain directions

Figure 6.16 Comparison of displacements in different grain directions



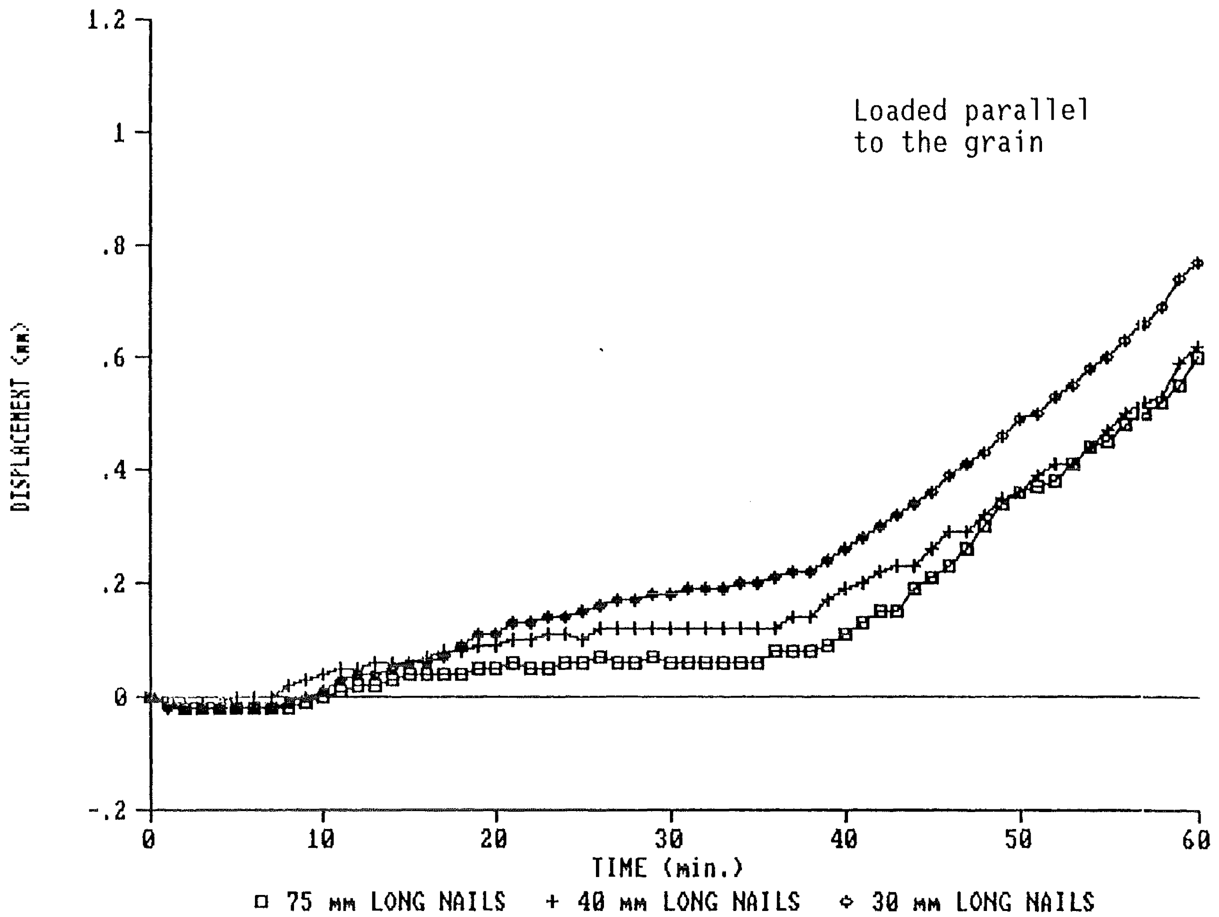
6.3.4 COMPARISON OF EFFECT OF NAIL LENGTHS AND NAIL DIAMETERS ON DISPLACEMENT BEHAVIOUR

To determine the effect of nail lengths on the performance of the joint, three different joints each having different nail length were tested. Plain shank nails of 75 mm length and galvanised nails of 40 mm and 30 mm lengths were used. The nominal nail diameter was 3.15 mm in all cases. The displacement results for a load of 2.31 kN applied both parallel as well as perpendicular to the grain are indicated in Figure 6.17. In the parallel direction only the shortest nail appears vulnerable and undergoes more displacement, while in the perpendicular direction both the shorter nails undergo same amount of displacement. Also note the greater initial displacement for 75 mm long nails in the perpendicular direction.

To observe the effect of nail diameter on joint displacement behaviour, 45 mm long galvanised nails of 4.35 mm diameter were also included in the tests. A load of 2.31 kN was applied parallel to the grain. A comparison of this result with that of 40 mm long galvanised nails of 3.15 mm diameter is indicated in figure 6.18. This figure suggests that nails of larger diameter undergo more deformation. This is the result of a single test only and more tests are necessary to confirm this behaviour.

6.3.5 SUMMARY OF 1/2 HOUR AND 1 HOUR DISPLACEMENTS

Table 6.1 shows the 1/2 hour and 1 hour displacements for all the tests carried out using steel gussets. For some forms of protection only a few load cases were investigated since their performance was very good (eg. two layers of 14.5 mm gibboard protection) or quite poor(eg. intumescent coating protection).



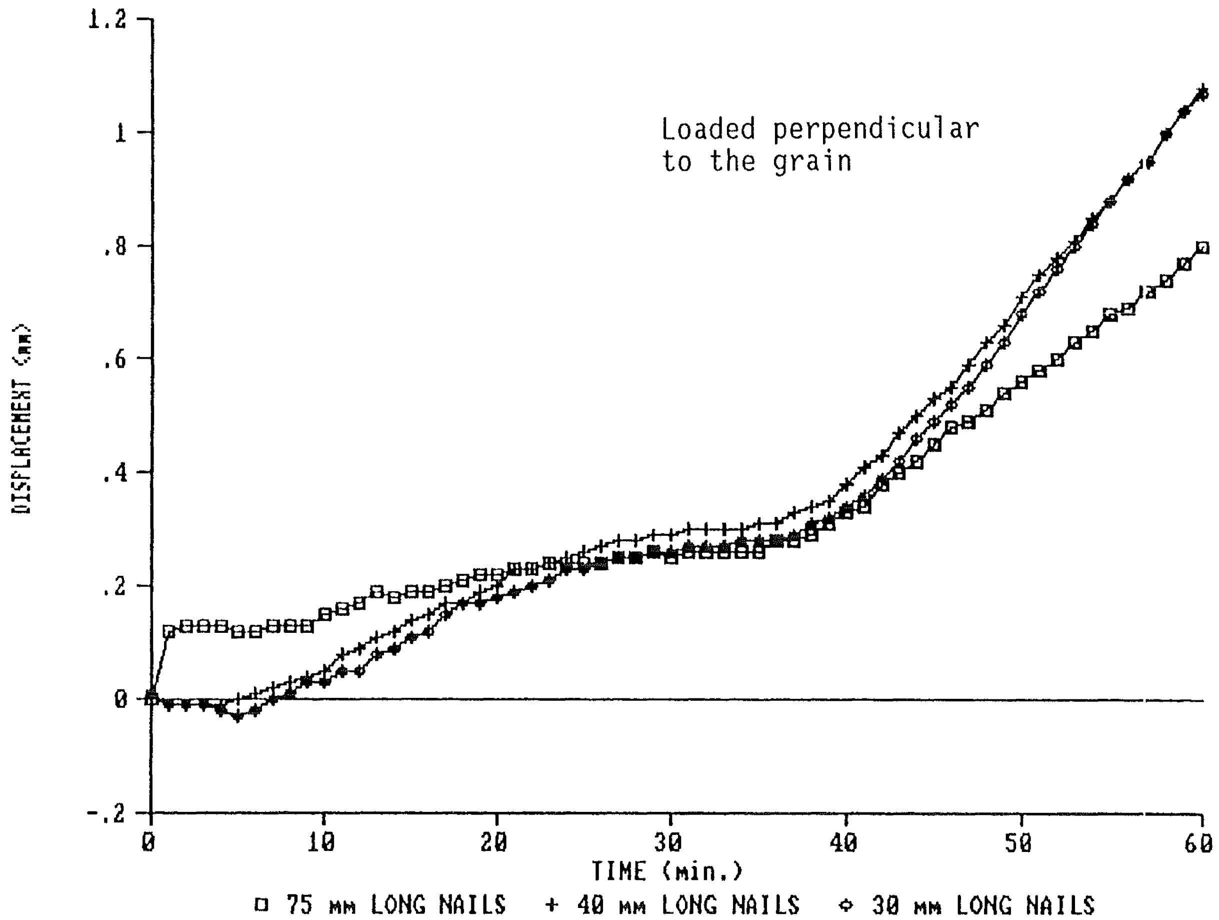


Figure 6.17 The effect of nail lengths on displacement behaviour

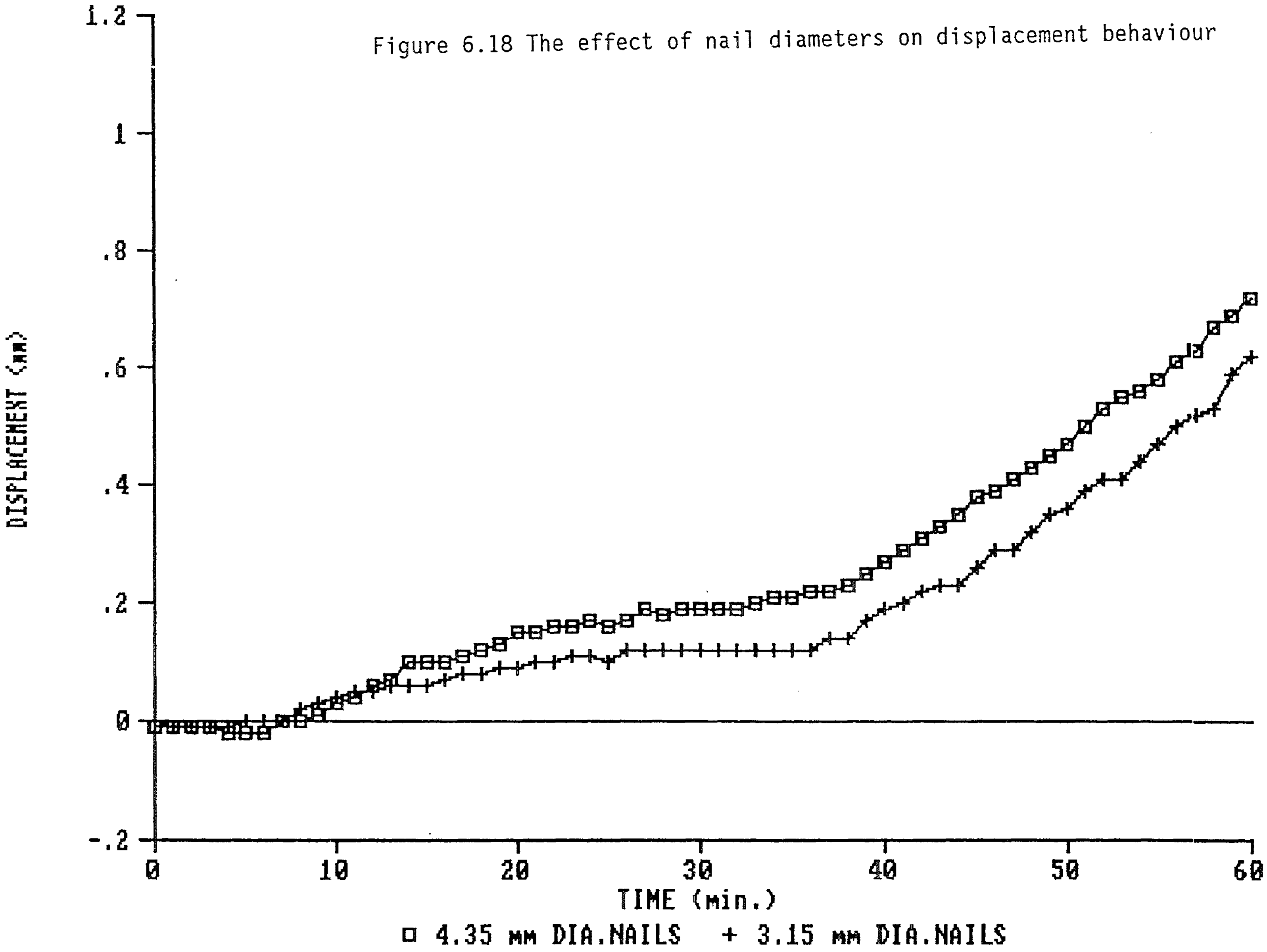


Figure 6.18 The effect of nail diameters on displacement behaviour
1.2
1
.8
.6
.4
.2
0
-.2
DISPLACEMENT (mm)
0
10
20
30
40
50
60
TIME (min.)
□ 4.35 mm DIA.NAILS + 3.15 mm DIA.NAILS

TABLE 6.1: ½ HOUR AND 1 HOUR DISPLACEMENTS IN MM UNDER FIRE CONDITIONS (5 MM STEEL GUSSET)

	LOAD ON SIX NAILS	PARALLEL ½ H	PARALLEL 1 H	PERPENDICULAR ½ H	PERPENDICULAR 1 H
19 mm Gibboard Protection:	0.58 kN	.1	.36	.08	.37
	1.16 kN	.1	.46	.16	.7
	1.73 kN	.03	.49	.11	.62
	2.31 kN				
	75 mm nails	.06	.6	.25	.8
	40 mm nails	.12	.62	.29	1.08
	30 mm nails	.18	.77	.26	1.07
	3.47 kN	.2	.76	.23	.92
	4.63 kN	.47	1.34	.4	.92
	7.87 kN	8.16 AT FAILURE		13	
2 Layers 14.5 mm Gibboard Protection:	1.16 kN	.13	.2	.08	.15
	1.73 kN	.04	.14	.17	.27
	2.31 kN				
	75 mm nails	.21	.28	.22	.54
	3.47	.47	.51	.31	.54
Intumescent Coating Protection:	1.16 kN	.39	.81	.65	1.21
	1.73 kN	.44	.97	.92	2.03
	2.31 kN	.69	1.81	1.13	2.01

6.4 LOAD-SLIP CHARACTERISTICS OF NAILS UNDER FIRE CONDITIONS FOR PLY WOOD GUSSETED JOINTS

The effect of loads and the influence of direction of loading, on load-deformation characteristics of nails in plywood gusseted joints, under fire conditions is discussed.

6.4.1 COMPARISON OF EFFECT OF LOADS ON JOINT DISPLACEMENT

Figure 6.19 indicates the effect of four different loads on joint displacement behaviour in both the grain directions. In the perpendicular direction, in all the load cases shown, the displacements begin to increase steadily after about 40 minutes of fire exposure. This is just past the stage when the moisture in the wood has been driven off and the nail temperatures increase accordingly. In the initial stages of the test, the joint exhibits similar displacement behaviour under both the lower loads. However with the higher loads the displacements begin to show after about 15 minutes of exposure to fire. The difference in final displacements at lower loads is also quite small compared to that at higher loads.

The behaviour in the parallel direction is quite similar to that of perpendicular direction.

6.4.2 COMPARISON OF DISPLACEMENTS IN DIFFERENT GRAIN DIRECTIONS

The displacements indicated by the joint when loaded either parallel to the grain or perpendicular to the grain was observed to be quite similar except for loads of 2.31 kN and 4.63 kN, in which cases the joint exhibited slightly more displacements in the parallel direction. The displacements observed for a load of 2.31 kN is indicated in figure 6.20.

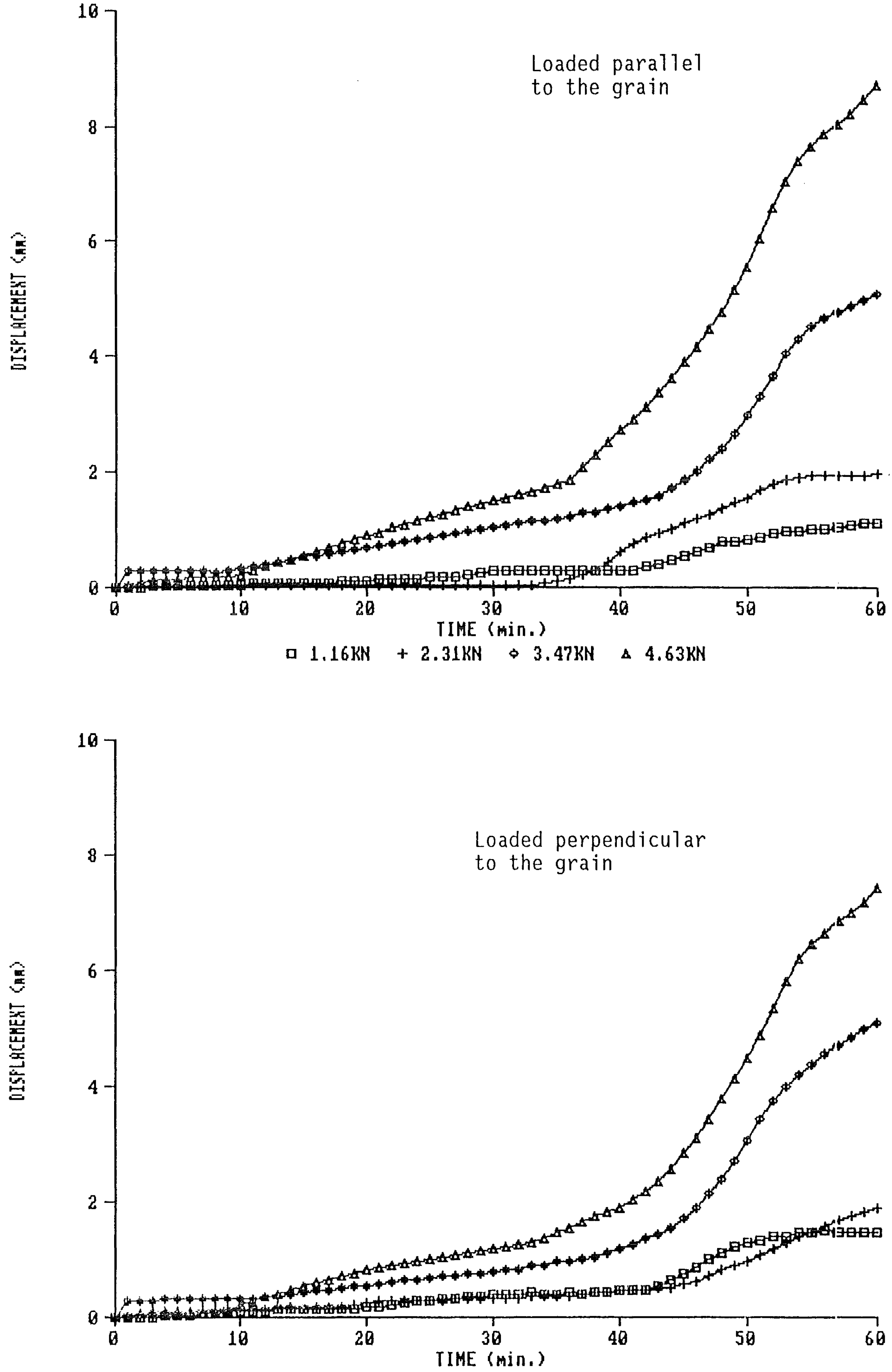
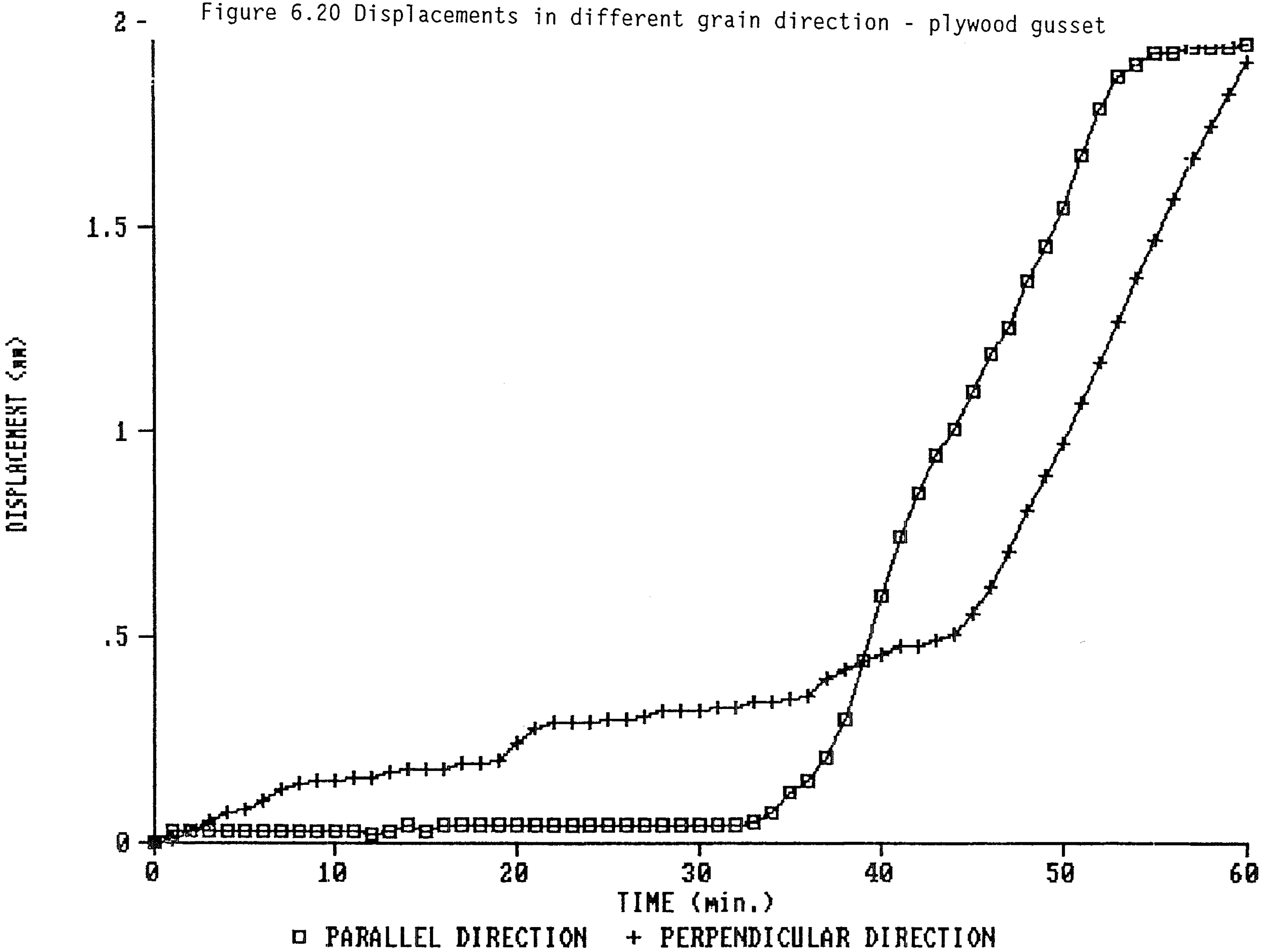


Figure 6.19 Effect of loads on joint displacement behaviour - plywood gusset

Figure 6.20 Displacements in different grain direction - plywood gusset



6.4.3 SUMMARY OF 1/2 HOUR AND 1 HOUR DISPLACEMENTS

These results are given in table 6.2

TABLE 6.2

LOAD (kN)	1/2 HR DISPLACEMENTS - mm		1 HR DISPLACEMENTS - mm	
	PARALLEL	PERPENDICULAR	PARALLEL	PERPENDICULAR
1.16	0.28	0.39	1.12	1.48
2.31	0.04	0.32	1.95	1.91
3.47	1.02	0.79	5.06	5.11
4.63	1.49	1.17	8.72	7.42

6.5 RESULTS OF NAIL TEMPERATURE MEASUREMENTS

The results of the tests conducted to determine the temperature that exists along the length of the embedded nail shank are presented.

6.5.1 TEMPERATURE DEVELOPMENTS AND TEMPERATURE GRADIENTS FOR NAILS IN A PROTECTED STEEL GUSSETED JOINT

Figure 6.21 shows the temperature development along the length of the embedded plain nail shank. A protection of 19 mm gibboard is simulated. Temperature developments are similar to the simulated temperature profile of the joint at measurement points closer to the nail head i.e., 10 mm and 20 mm. However towards the tip of the nail, the temperature development is delayed. This confirms the existence of a thermal gradient along the length of the nail shank.

Figure 6.22 compares the 1/2 and 1 hour temperature gradients. The nail tip temperatures are quite low compared to the nail head temperatures. The rate of temperature increase is proportional to the heat input as indicated by the slope of the temperature gradients.

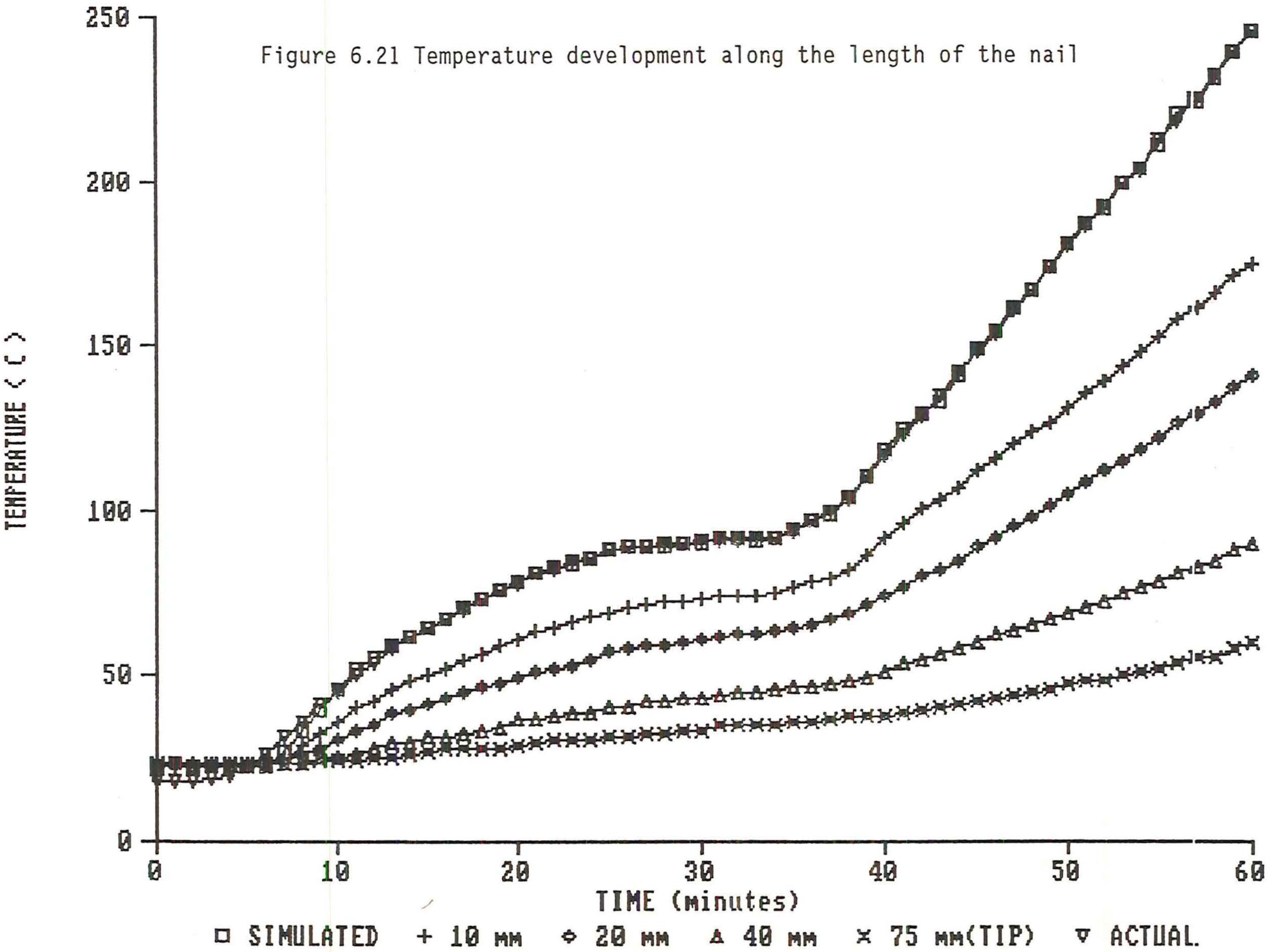


Figure 6.21 Temperature development along the length of the nail

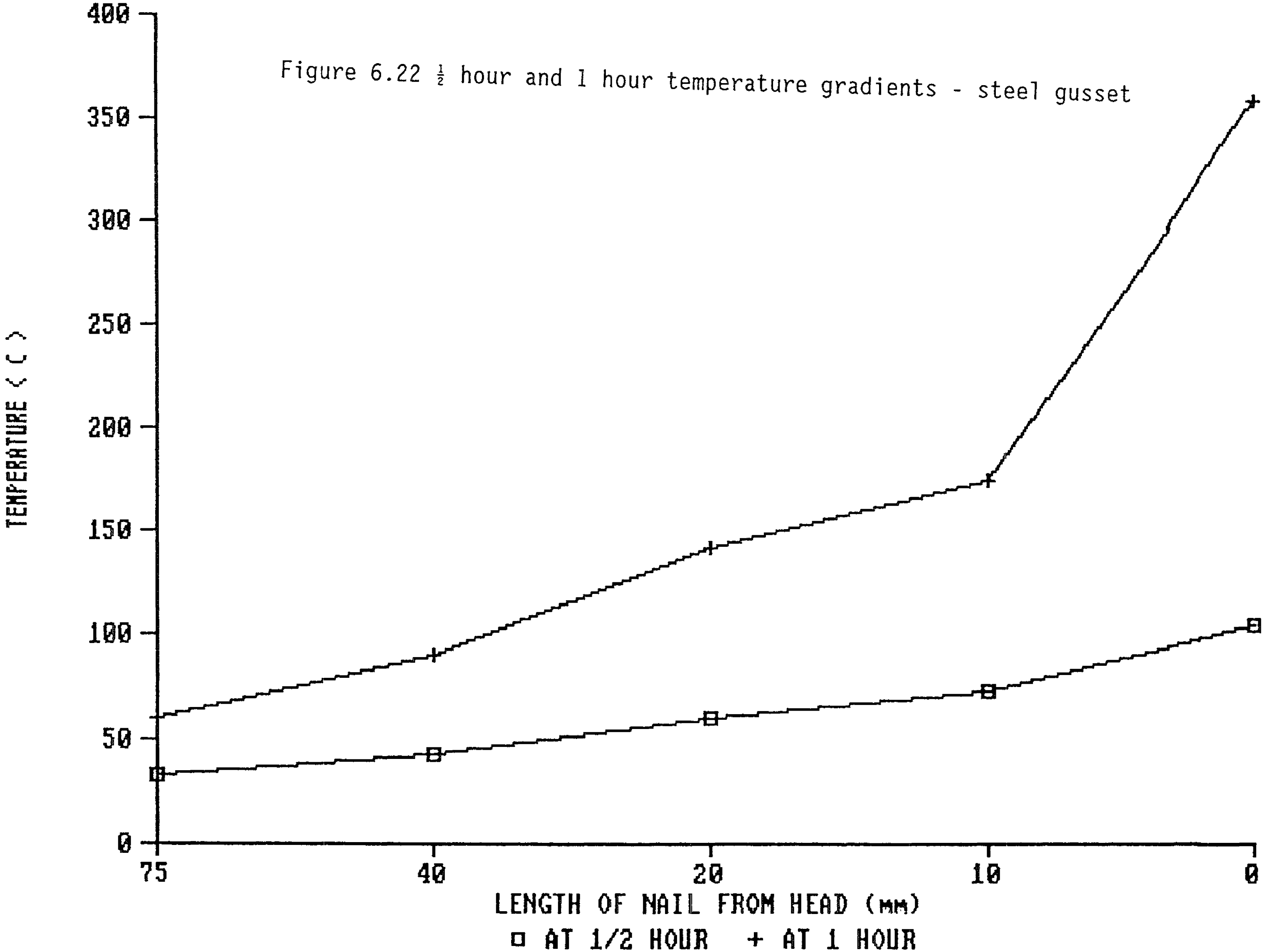


Figure 6.22 ½ hour and 1 hour temperature gradients - steel gusset

6.5.2 TEMPERATURE DEVELOPMENTS AND TEMPERATURE GRADIENTS FOR NAILS IN A PROTECTED PLYWOOD GUSSETED JOINT

Figure 6.23 indicates the temperature development along the length of the 85 mm long gun nail. The reference temperatures are also shown. The plywood gusset provides much better insulation to the nails than the steel gusset did. The maximum temperature in the nail at the underside of the gusset is less than 100^{o}C which indicates that the nails can still maintain their rigidity and transfer the load safely (Hillis and Rozsa, 1978).

Figure 6.24 shows the 1/2 and 1 hour temperature gradients. The slower thermal build-up for nails in this type of joint is clearly indicated.

6.5.3 COMPARISON OF TEMPERATURE GRADIENTS OF NAILS IN DIFFERENT GUSSET MATERIALS

Figure 6.25 compares the 1/2 and 1 hour temperature gradients for plywood and steel gusseted joints. Temperatures along the nail in plywood gusseted joint are consistently lower than that under steel gusseted joint.

6.6 THE EFFECT OF GLULAM SURFACE TEMPERATURE ON JOINT DISPLACEMENT

The temperatures on the glulam surface in a protected gusseted joint and the effect of it in joint displacement when loaded under fire conditions is discussed.

Figure 6.26 is the plot of displacements at 1/2 hour and 1 hour versus temperature on the glulam surface for a load of 1.8 times basic nail load applied on a joint with 5 mm steel gusset and three different types of protection, i.e. 2 layers of 14.5 mm gibboard, 1 layer of 19 mm

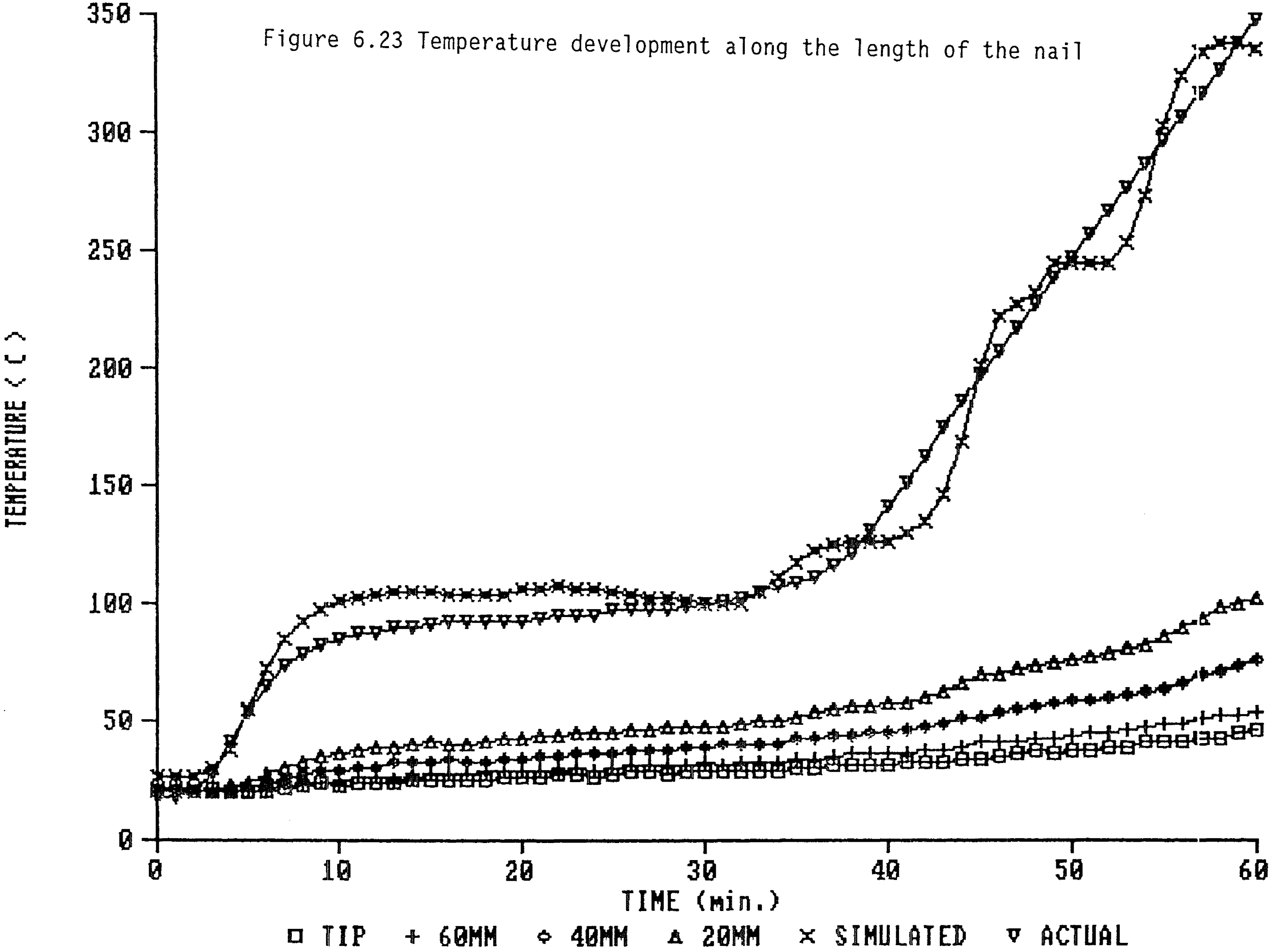


Figure 6.23 Temperature development along the length of the nail

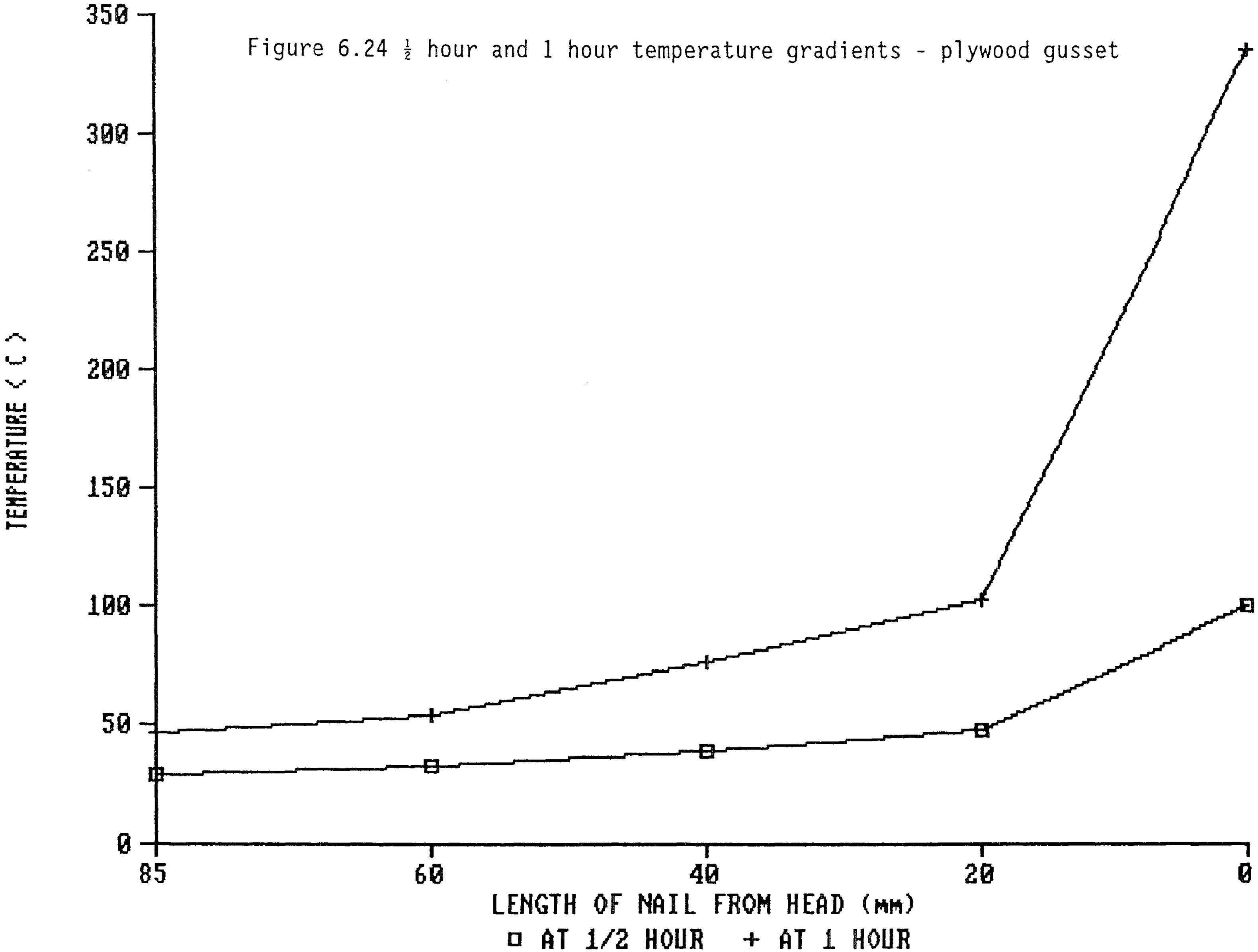
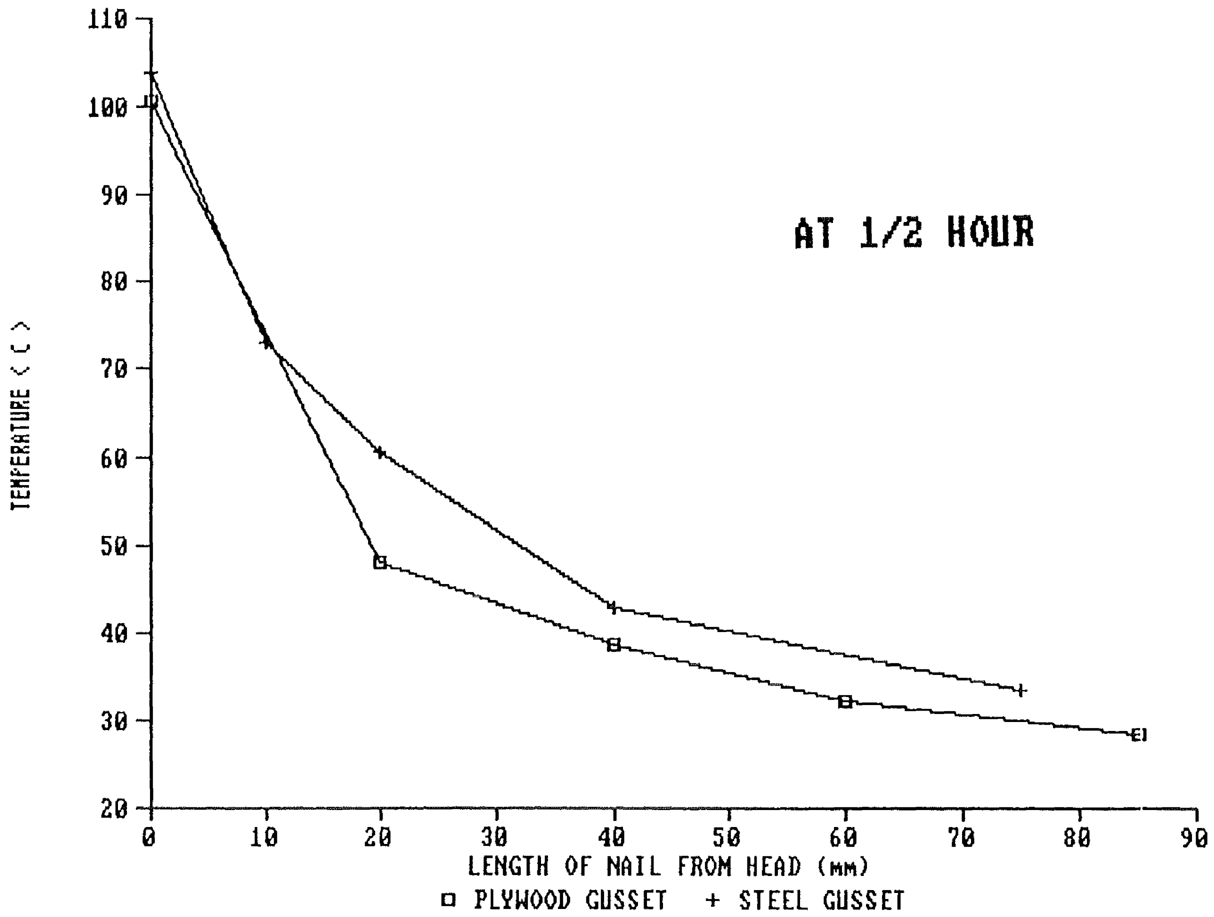


Figure 6.24 ½ hour and 1 hour temperature gradients - plywood gusset
TEMPERATURE (C)
350
300
250
200
150
100
50
0
85
60
40
20
0
LENGTH OF NAIL FROM HEAD (mm)
□ AT 1/2 HOUR + AT 1 HOUR



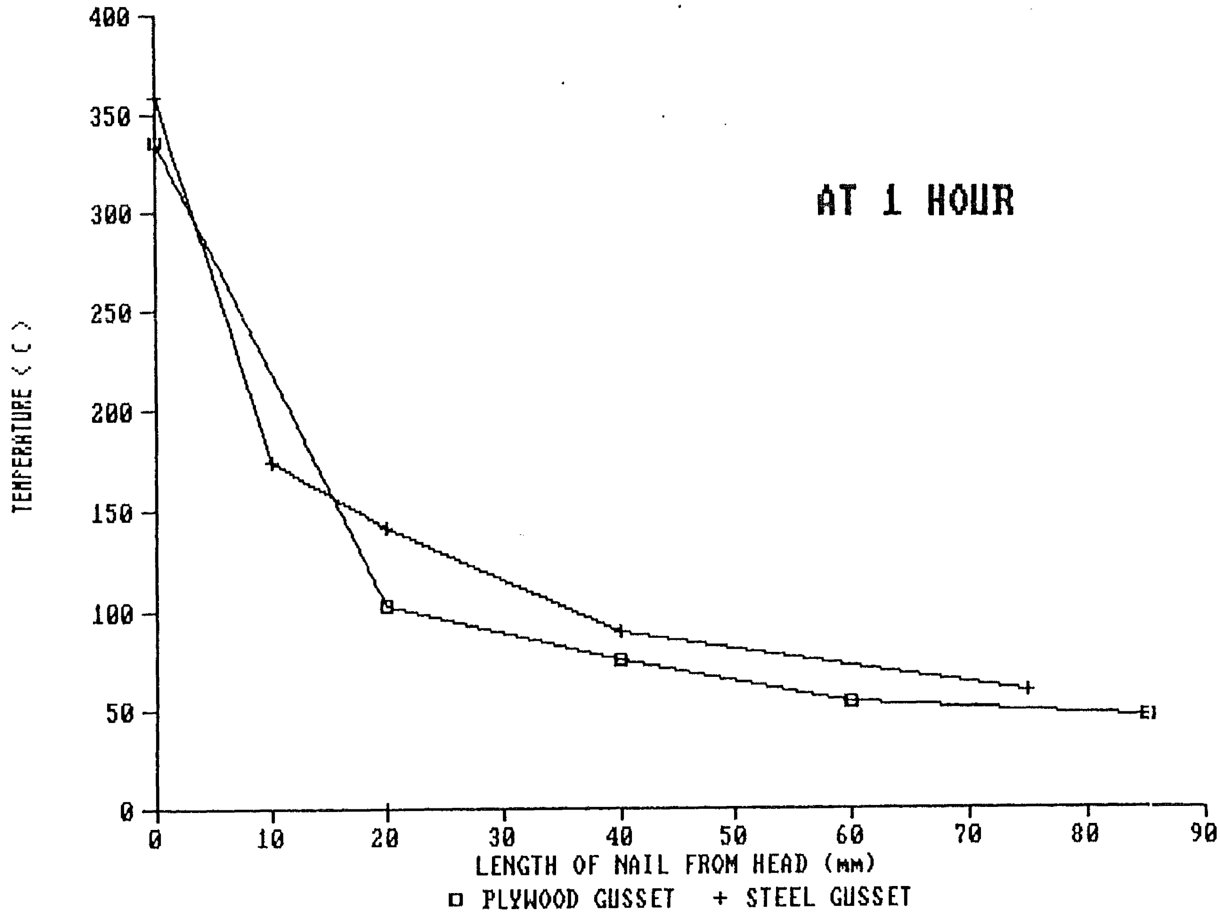
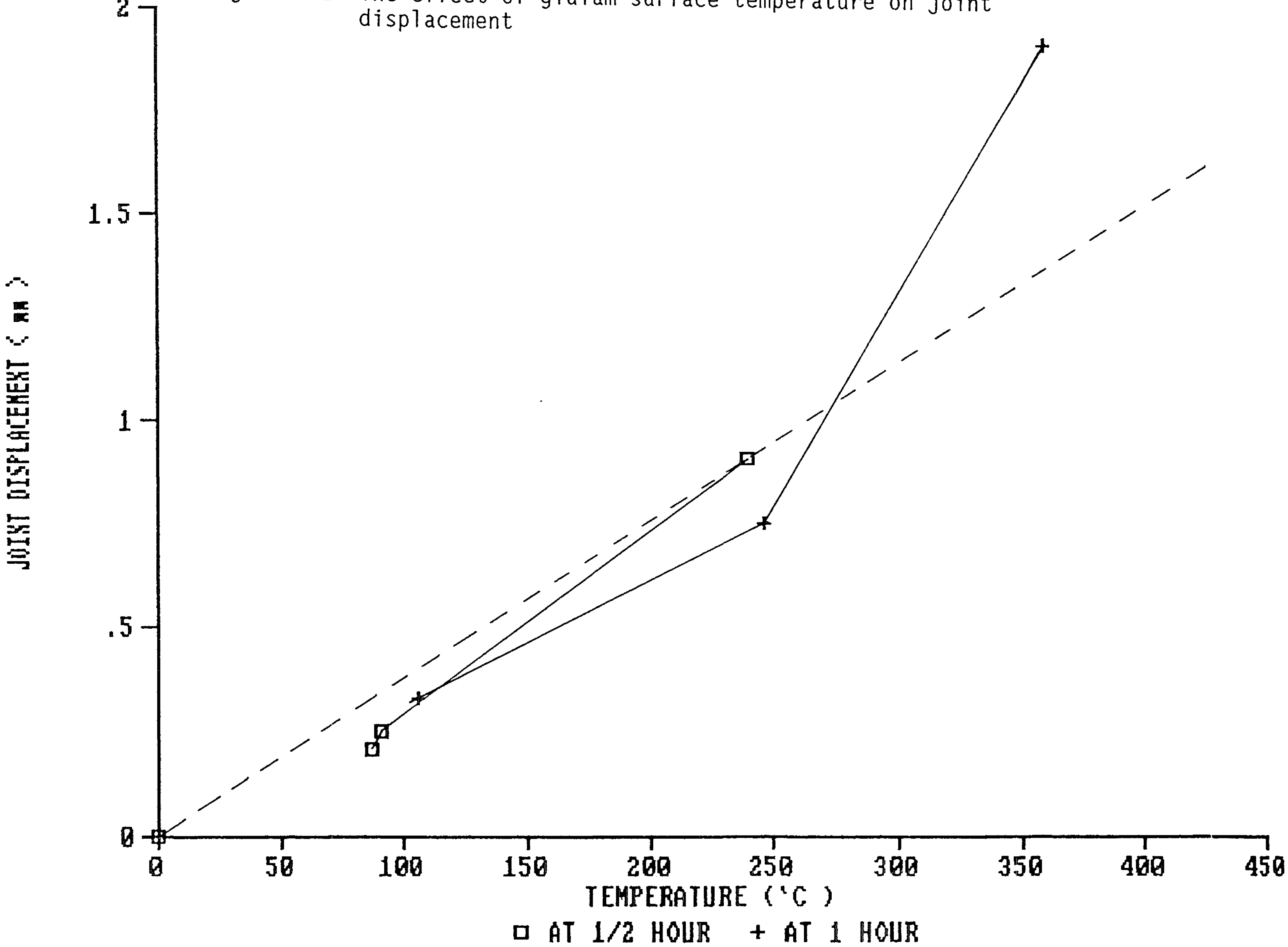


Figure 6.25 comparison of ½ hour and 1 hour temperature gradients for steel and plywood gusseted joints

Figure 6.26 The effect of glulam surface temperature on joint displacement



gibboard and intumescent coating. An approximate relationship between temperature and displacement is indicated by the dotted lines.

This relationship would be useful to predict the displacement of the joint that could be expected, simply by knowing the temperature on the glulam surface from an unloaded fire test for any type of protection.

There isn't enough data to develop a similar relationship for plywood gusseted joints.

6.7 COMPARISON OF THE LOAD-DEFORMATION BEHAVIOUR OF THE JOINT UNDER COLD AND FIRE CONDITIONS FOR THE STEEL GUSSETED JOINT

Figure 6.27 shows the load-deformation behaviour of the steel gusseted joint under cold and fire conditions. The simulated protection under fire conditions is 19 mm gibboard.

Under cold conditions the joint was loaded at a loading rate of 0.5 mm/min. The joint reached a maximum load of 13.98 kN for a displacement of about 4 mm, before it failed due to nail heads shearing off.

Under fire conditions a constant load of 4.63 kN was maintained for 1 hour producing a deflection of 1.34 mm and then the joint was loaded to failure. The loading to failure took about 1 minute. In this case the joint carried a maximum load of 7.87 kN for a displacement of about 8 mm before it failed.

It is interesting to note that the maximum load carried by the joint is halved under fire conditions while the displacement at the maximum load is doubled.

6.8 RESULTS OF TESTS TO DETERMINE THE BENDING STRENGTH OF NAILS

Table 6.3 indicates the bending strength calculations for different nails. Three nails were tested under each nail diameter. Both gun nails

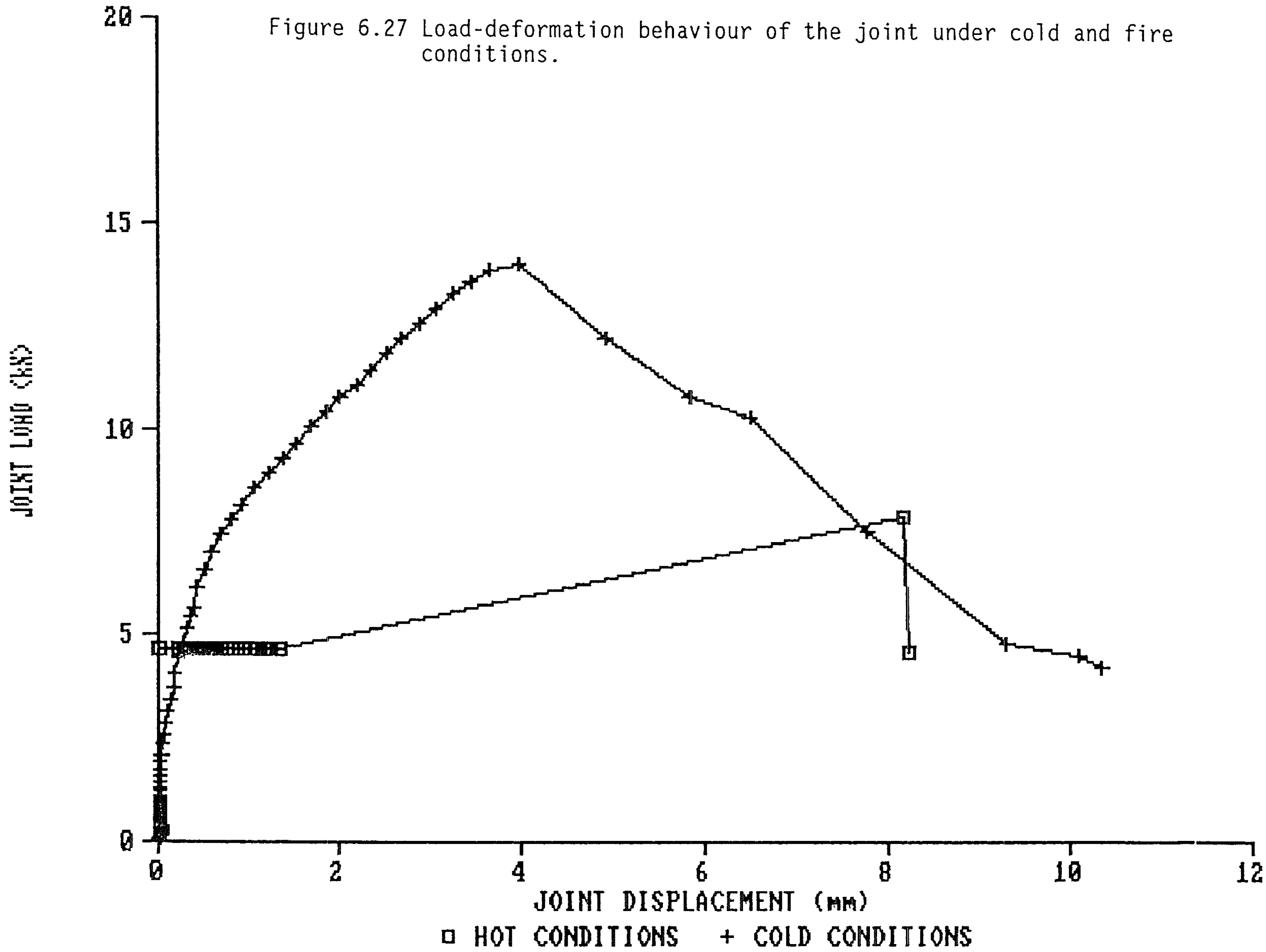


Figure 6.27 Load-deformation behaviour of the joint under cold and fire conditions.
20
15
10
5
0
JOINT LOAD (kN)
0
2
4
6
8
10
12
JOINT DISPLACEMENT (mm)
□ HOT CONDITIONS + COLD CONDITIONS

and plain nails were tested. The nail diameter, the maximum load sustained by the nail, the plastic moment of the nail, the plastic section modulus of the nail and the maximum bending strength of the nail are indicated in the respective columns.

As shown in the table, 3.15 mm diameter plain shank nails possess the highest strength. Gun nails appear to have lower bending strength compared to plain shank nails.

TABLE 6.3 : NAIL BENDING STRENGTH CALCULATIONS

(d_n) mm	P N	M_p = .25Pxl N.mm	$Z_p = d_n^3/6$ mm^3	f Mpa	f_{ave} Mpa
3.33	295	4425	6.15	719.5	
3.33	290	4350	6.15	707.3	711
3.33	290	4350	6.15	707.3	
3.15	325	4875	5.21	935.7	
3.15	300	4500	5.21	863.7	888
3.15	300	4500	5.21	863.7	
3.35	340	5100	6.27	813.4	
3.35	325	4875	6.27	777.5	789
3.35	325	4875	6.27	777.5	

CHAPTER SEVEN

A RELATIONSHIP FOR LOAD-SLIP CHARACTERISTICS OF NAILS AT ELEVATED TEMPERATURES

7.1 INTRODUCTION

Relationships for load-slip behaviour of nails, in steel and plywood gusseted joints are expressed in this chapter. Both cold and fire conditions are considered. The joint is protected by a 19 mm gibboard under fire conditions. For fire conditions, it is not possible to perform a load-slip test at a particular point in time. So the load-slip relationship has been derived using the results of constant load tests.

To develop an expression for the load-slip behaviour of nails after a particular period of fire exposure, say one hour, the deflections of the constant load tests after one hour are plotted against the corresponding loads, to provide an implied load-slip relationship. The method is described in more detail below.

7.2 STEEL GUSSETED JOINTS

The gusset is 5 mm thick steel. The load-slip expressions indicated are for 75 mm long, 3.15 mm nominal diameter plain shank nails, which had an embedment length of about 70 mm.

7.2.1 LOAD-SLIP RELATIONSHIP UNDER COLD CONDITIONS

The load-deformation relationship outlined here is the one proposed by Foschi (Foschi, 1974), which has the general form as shown in figure 7.1. The curve can be represented in an exponential form according to,

$$P = (P_0 + P_1 w)\ (1 - \exp(-kw/P_0))$$

The determination of various parameters from the experimental load-

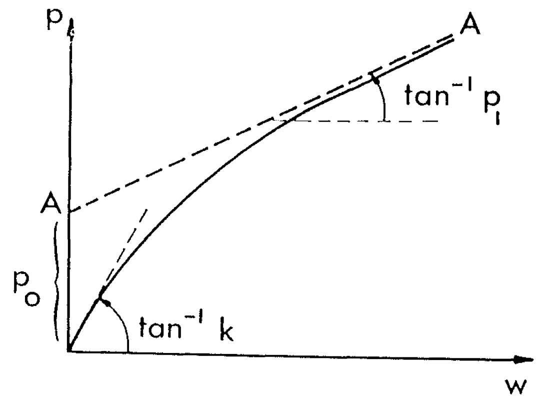


Figure 7.1 Load-deformation relationship for wood

deformation curves are indicated in figure 7.2. The values of relevant parameters are,

P_0 = 0.96 kN

P_1 = 0.405 kN/mm

k = 6.06 kN/mm

Therefore the load-deformation relationship is,

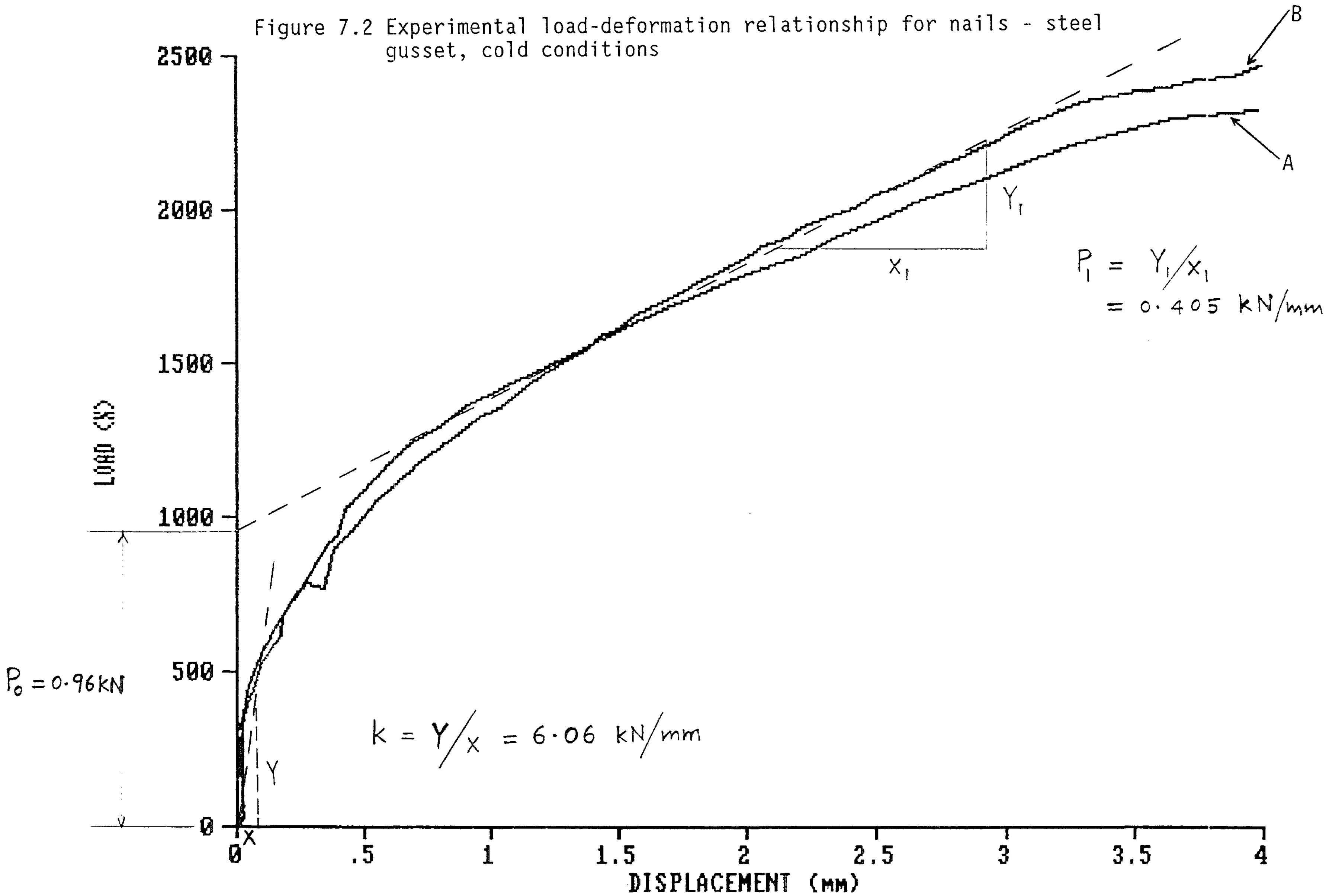
$P = (0.96 + 0.405w)\ (1 - \exp\ (-6.06w\ /\ 0.96))$

Very good agreement can be seen between the curve fitted according to the above expression and the experimental curves, as shown in figure 7.3.

7.2.2 LOAD-SLIP RELATIONSHIP UNDER FIRE CONDITIONS

The same exponential relationship as used under cold conditions is extended to fit a curve for the experimental results of the load-deformation behaviour of nails under fire conditions, shown in figure 7.4. Each experimental point corresponds to deflection at the end of one hour in a constant load test. The deflections in both parallel and perpendicular to the grain are shown. The procedure for determining the

Figure 7.2 Experimental load-deformation relationship for nails - steel gusset, cold conditions



A - PARALLEL LOAD B - PERPENDICULAR LOAD

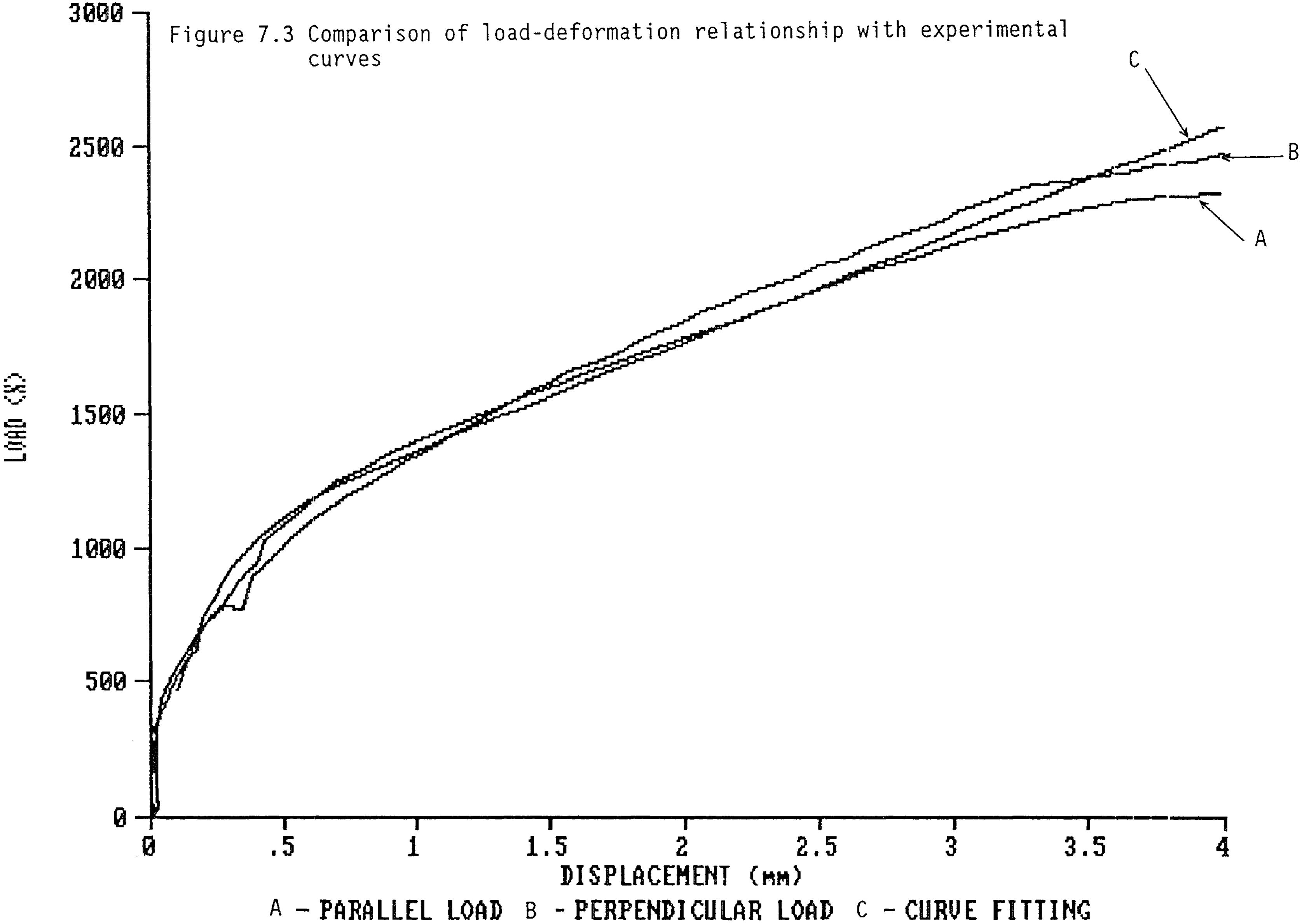


Figure 7.3 Comparison of load-deformation relationship with experimental curves

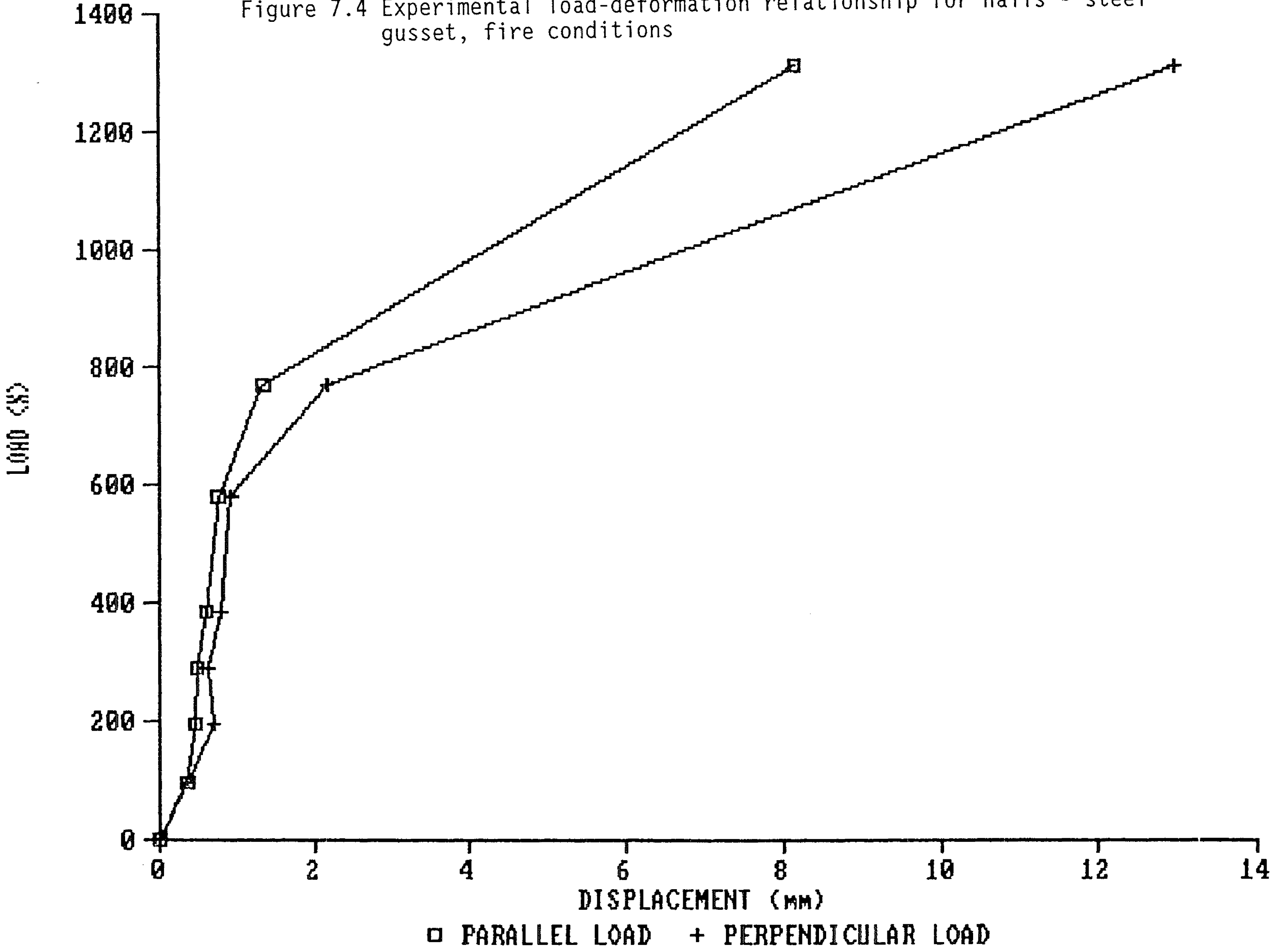
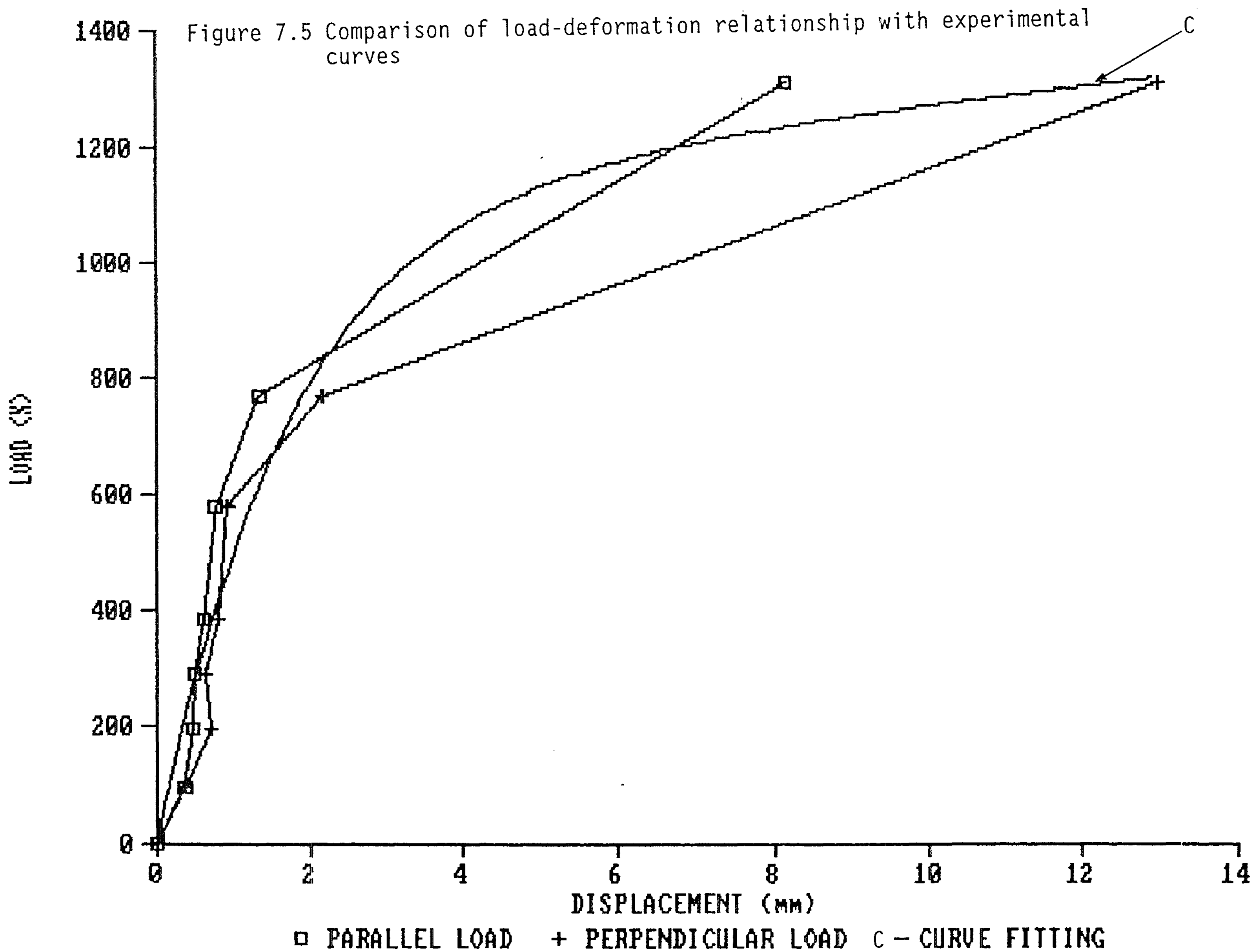


Figure 7.4 Experimental load-deformation relationship for nails - steel gusset, fire conditions

Figure 7.5 Comparison of load-deformation relationship with experimental curves



relevant parameters is similar to that used for cold conditions.

The values of these parameters are,

P_0 = 1.119 kN

P_1 = 0.016 kN/mm

k = 0.65 kN/mm

The resulting load-deformation expression is,

$P = (1.119 + 0.016w)\ (1 - \exp(-0.65w/1.119))$

The load-slip curve according to the above expression is compared against the experimental results in figure 7.5. The curve is a good representation of the load-slip behaviour.

7.3 PLYWOOD GUSSETED JOINTS

The load-slip relationships for 85 mm long gun nails under cold and fire conditions are derived. The nails had an embedment length of 65 mm.

7.3.1 LOAD-SLIP RELATIONSHIP UNDER COLD CONDITIONS

Foschi and Bonac (1977) observed no significant difference in the load-slip behaviour among the cases of load parallel to the face grain, load perpendicular to the face grain or at 45^0. The exponential form of load-deformation relationship indicated by them is used to establish a relationship for the experimental points shown in figure 7.6.

The values of the various parameters in this case are,

P_0 = 1.0 kN

P_1 = 0.165 kN/mm

k = 5.15 kN/mm

Hence, the load-deformation relationship takes the form,

$P = (1.0 + 0.165w)\ (1 - \exp(-5.15w/1.0))$

The curve represented by the above expression very closely follows

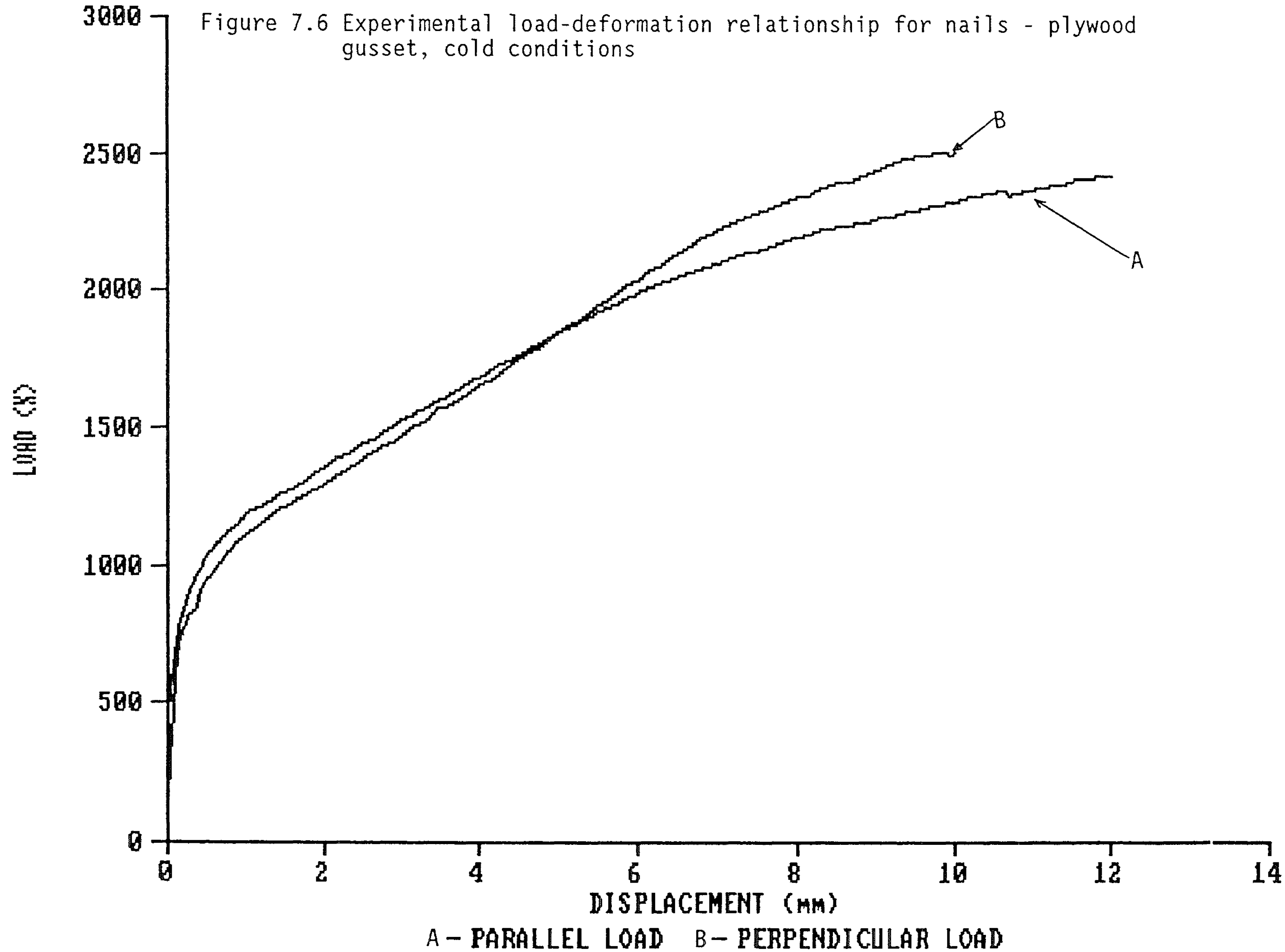


Figure 7.6 Experimental load-deformation relationship for nails - plywood gusset, cold conditions

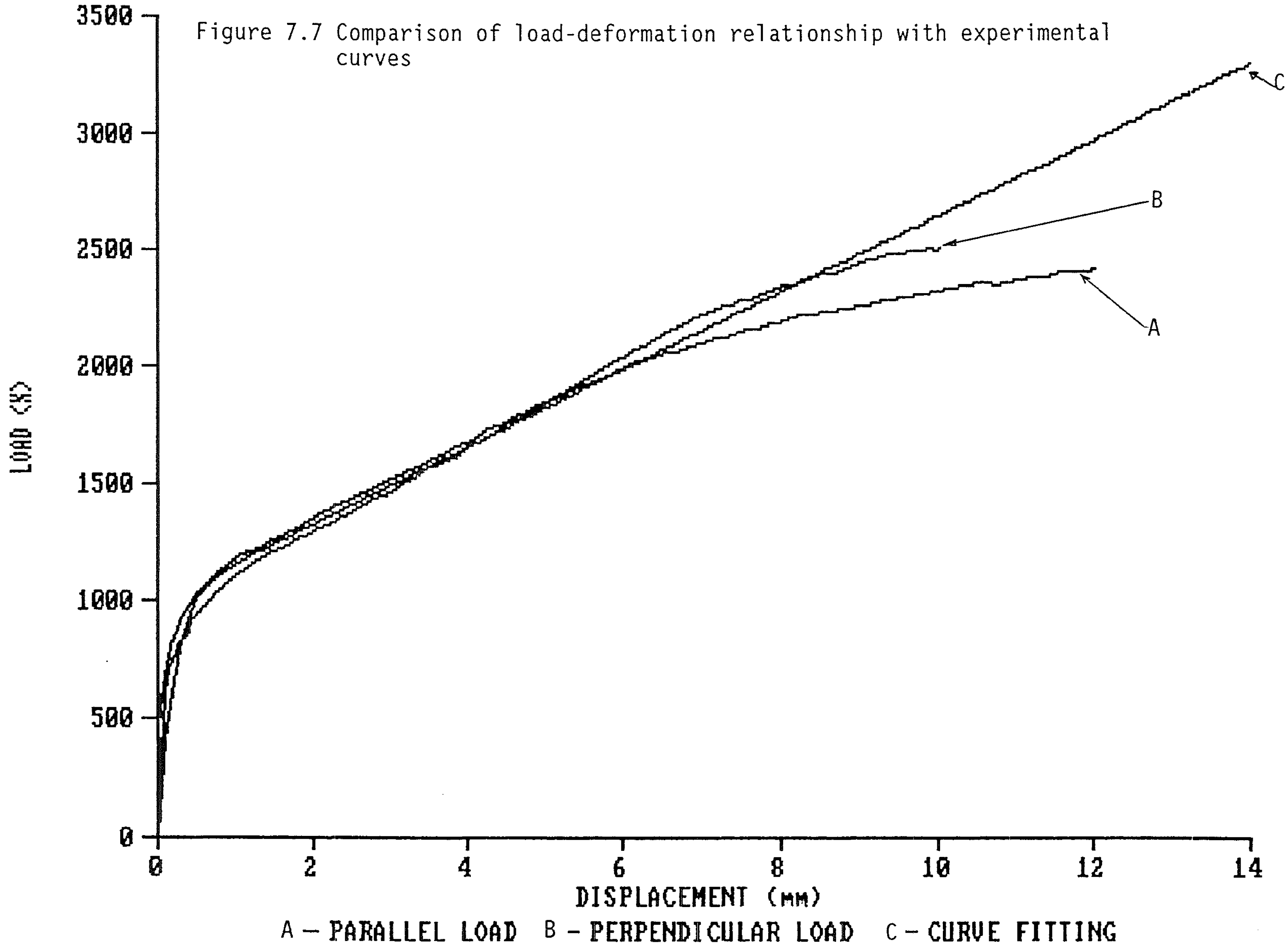
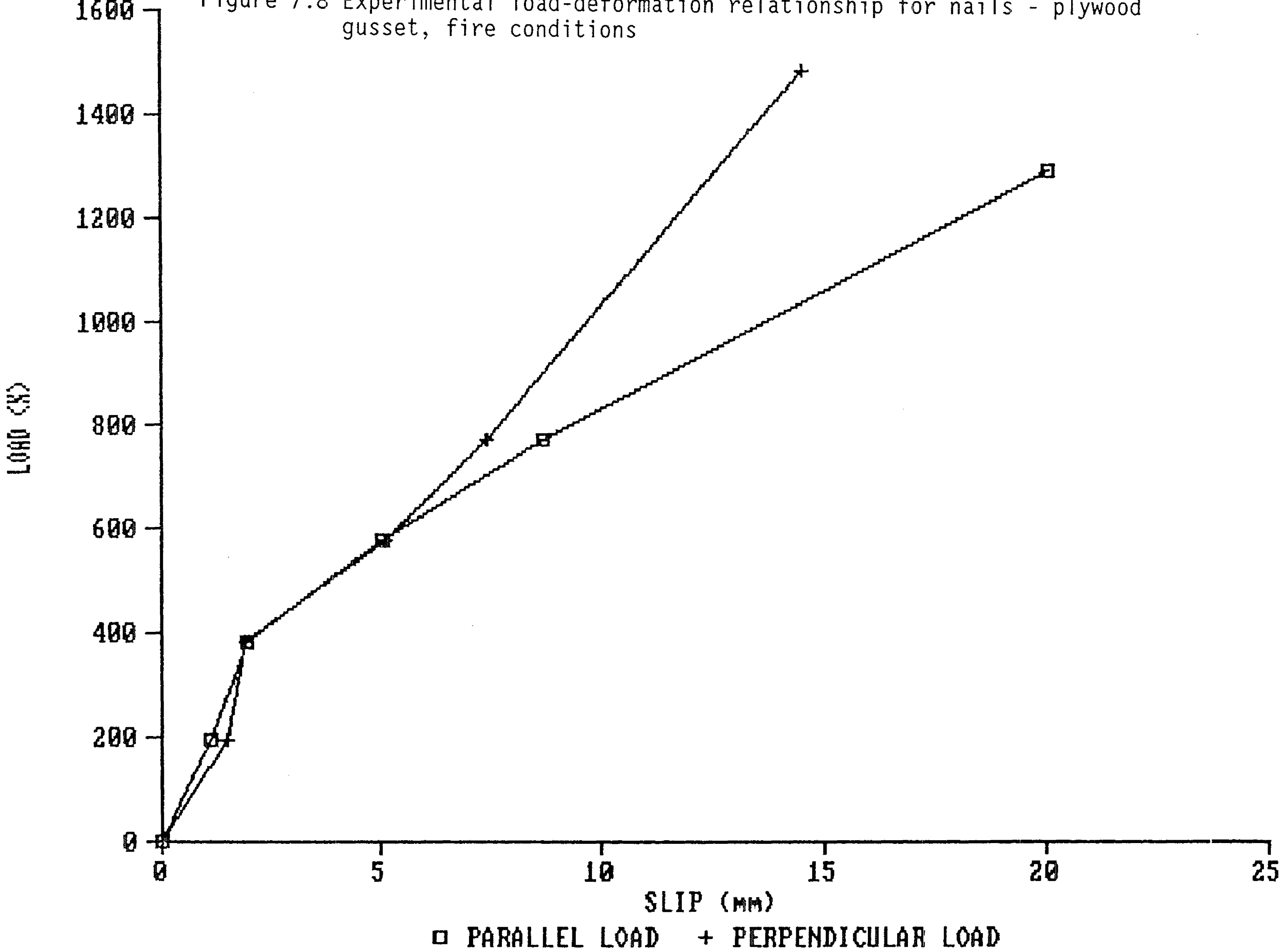


Figure 7.7 Comparison of load-deformation relationship with experimental curves

Figure 7.8 Experimental load-deformation relationship for nails - plywood gusset, fire conditions



the experimental results upto a slip of about 7 mm as shown in figure 7.7.

7.3.2 LOAD-SLIP RELATIONSHIP UNDER FIRE CONDITIONS

The experimental load-slip curves are indicated in figure 7.8. A similar method as used with steel gussets is used to determine the values for various parameters. The values of these parameters are,

P_0 = 0.28 kN

P_1 = 0.06 kN/mm

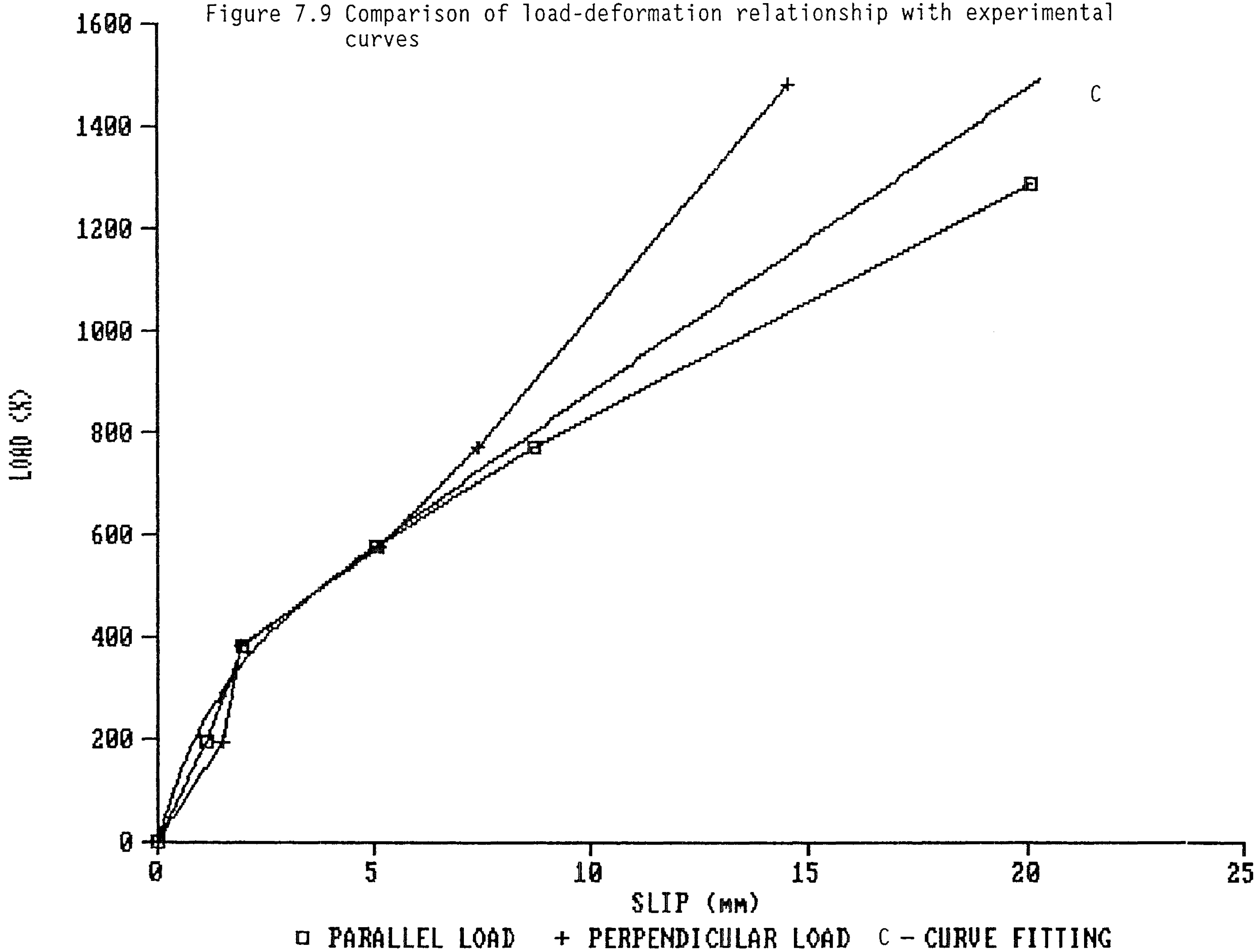
k = 0.3 kN/mm

Therefore the load-deformation relationship is,

$P = (0.28 + 0.06w)\ (1\text{-}\exp(-.3w/.28))$

Experimental load-deformation relationship is well represented by this expression as shown in figure 7.9.

Figure 7.9 Comparison of load-deformation relationship with experimental curves



CHAPTER EIGHT

COMPUTER MODEL

8.1 INTRODUCTION

A computer model which uses the load-deformation relationships developed in the previous chapter to determine the moment-rotation characteristics of the joint, is discussed. This model was originally developed by Thurston and Flack (1979) and later modified by Dr. P. J. Moss in the Department of Civil Engineering at the University of Canterbury. Considerable project time was saved by choosing this model and modifying it to suit our purposes, rather than developing a special purpose one. In order to get a complete history of nail forces and nail slips at higher moments and rotations, some modifications were added to the model.

8.2 DESCRIPTION OF THE MODEL

A brief description of the underlying assumptions, the input parameters to the model, the workings of the model and the output results obtained is given in the following paragraphs.

8.2.1 ASSUMPTIONS

The application of this model in our case, was to analyse the knee joint of a portal frame. In using this model it was assumed that the maximum rotation of the joint takes place at the rafter nail group and thus the rotation of the joint is the rotation due to this nail group only. This assumption might underestimate the rotation of the joint because even though the moment at the bottom nail group (column nail group) may be smaller, there will be some rotation there, which is not taken into account. Equal spacing between rows of nails in the parallel and perpendicular directions to the grain is another requirement dictated by the present model. However this could easily be altered in future, if circumstances warrant it.

8.2.2 INPUT PARAMETERS TO THE MODEL

These are,

(1) The breadth and depth of nailing pattern

(2) The spacing of nails along each row (i.e. pitch)

(3) Number of rows of nails

(4) Manner of nailing, whether staggered or unstaggered

(5) The load-deformation relationship for the nails

8.2.3 WORKING OF THE MODEL

(1) From the breadth and the depth of the nailing pattern, the program first calculates the radial distances of the nails from the centroid. The centroid is taken as the centre of the nail group geometry.

(2) In the next step, small increments of rotations are generated.

(3) Knowing the rotation (θ) and the radial distances (R) of the nails, the nail slip is worked out from the relationship,

$S = R\theta$.

(4) Knowing the nail slip, the program calls a subroutine which computes the force in the nail, depending on the load-deformation relationship incorporated in the model.

(5) The joint moment is then worked out by taking moments of the nail forces about the centroid of the nail group and summing them up.

8.2.4 OUTPUT RESULTS

As output the program gives the total number of nails, their location, the moment rotation characteristics of the joint, the maximum nail force under the applied loading and the corresponding nail slip.

CHAPTER NINE

APPLICATION OF MODEL RESULTS- PORTAL FRAME DESIGN EXAMPLE

9.1 INTRODUCTION

This chapter analyses the structural behaviour of a portal frame exposed to fire. The portal frame is 3 pinned and supports the roof of an industrial building. The ceiling of the roof is assumed to be fire-resistant. A charring rate of 0.6 mm/min is assumed in calculating the fire performance of exposed rafter and column surfaces, as stated in MP9 (SANZ, 1987). The knee joint of the portal is analysed considering both nailed-on steel and plywood gussets. The load-slip curves developed in chapter 7 are used for the analysis of the joint. This information, together with the joint details are input into the computer program to compute the moment-rotation characteristics of the joint. Thus the rotation of the joint corresponding to the design moment during fire is obtained. This is then used to predict the deflections of the knee and apex of the portal frame.

9.2 BUILDING DETAILS

Building size : 18 m x 30 m

Portal span : 18 m

Bay spacing : 5 @ 6 m   Figure 9.1 shows a view of the building.

9.3 LOADS

Dead load = 0.3 kPa

Snow load = 0.375 kPa

Dead + Snow load = 0.675 kPa

Wind load : (wind speed = 40 m/s, see figure 9.2 for details)

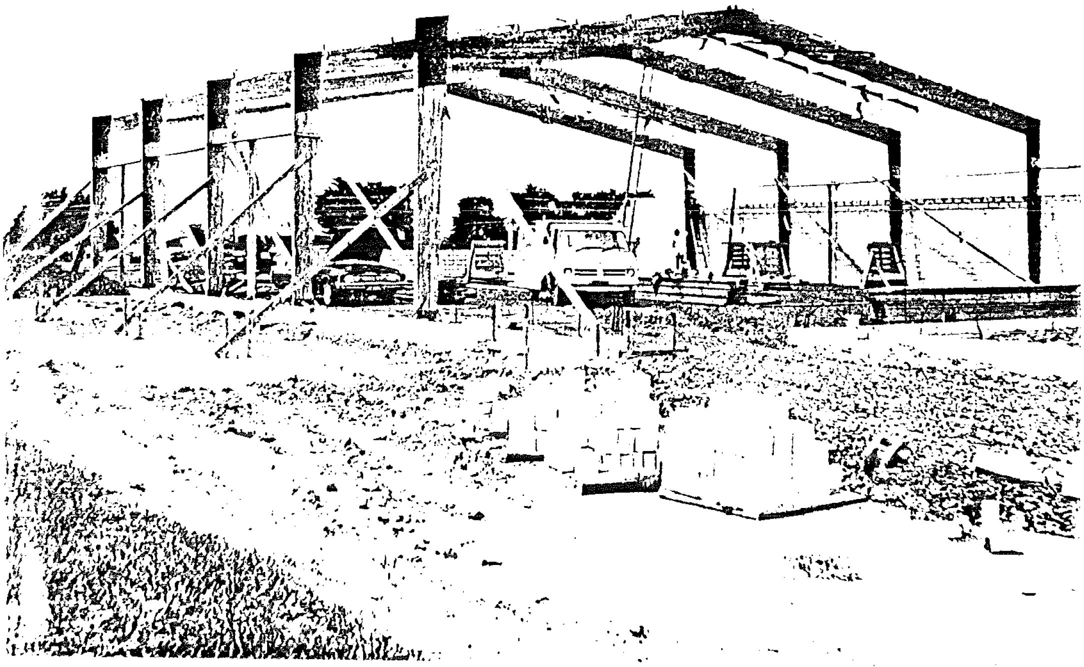


Figure 9.1 Portal framed building

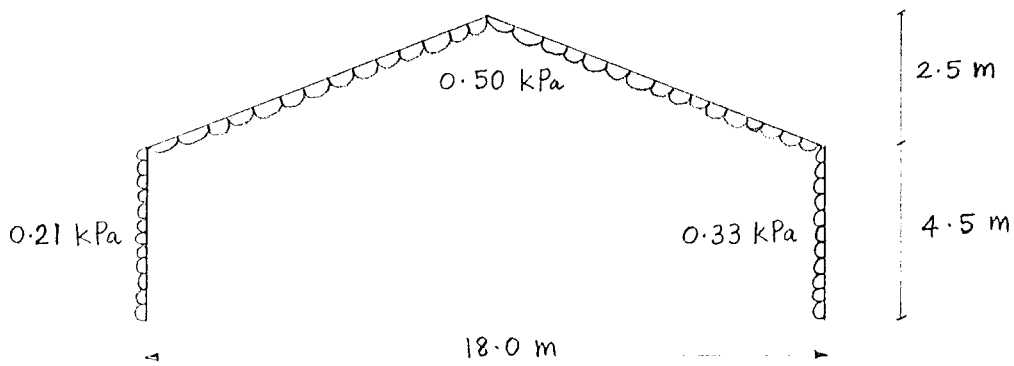
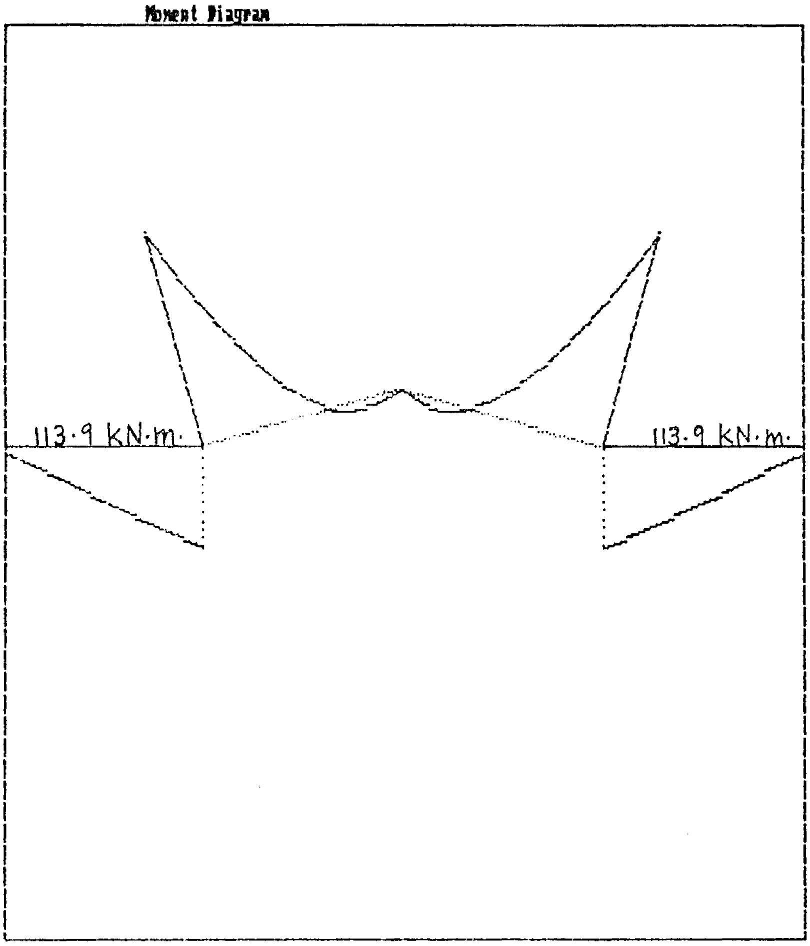


Figure 9.2 Wind loads on the frame

9.4 PORTAL DESIGN

Joint moments obtained from computer analysis are shown below.



	(dead + snow)	(dead + fire)
Maximum moment,	M (kN. m)	M (kN. m)
from analysis	= 113.9	= 0.3x113.9/0.675 = 50.6
Basic working		
stress, F_b (Mpa)	= 10.6	= 10.6 (NZS 3603:1981)
Duration of load		
factor, K_1	= 1.4 (Table 3 1989 amendment to NZS 3603:1981)	= 2.0 MP9 (SANZ, 1987)

$F_b = K_1F'_b = 10.6 \times 1.4 \qquad = 14.84$ Mpa (NZS 3603:1981)

$Z_{required} \qquad = 113.9 \times 10^6/14.84 \quad = 7.67 \times 10^6 mm^3$

Assume breadth of portal, b = 135 mm

Hence $\qquad d = (7.67 \times 10^6 \times 6/135)^{0.5} = 584$ mm

Use $\qquad d = 585$ mm (13 numbers of 45 mm laminations)

$Z_{provided} = 7.70 \times 10^6 mm^3 \; > Z_{required}$

9.5 FIRE RESISTANCE CHECK FOR PORTAL FRAME

Under fire conditions MP9 (SANZ, 1987) states that only dead load need be considered. The moment under fire conditions (dead load only) as worked out earlier, $M_{fire} = 50.6$ KN. m

Section properties (A charring rate of 0.6 mm/min. is assumed)

	before fire	after fire
b	135 mm	63 mm (charring two sides)
d	585 mm	549 mm (Charring on bottom)
Z	$7.70 \times 10^6 mm^3$	3.16×10^6 mm^3

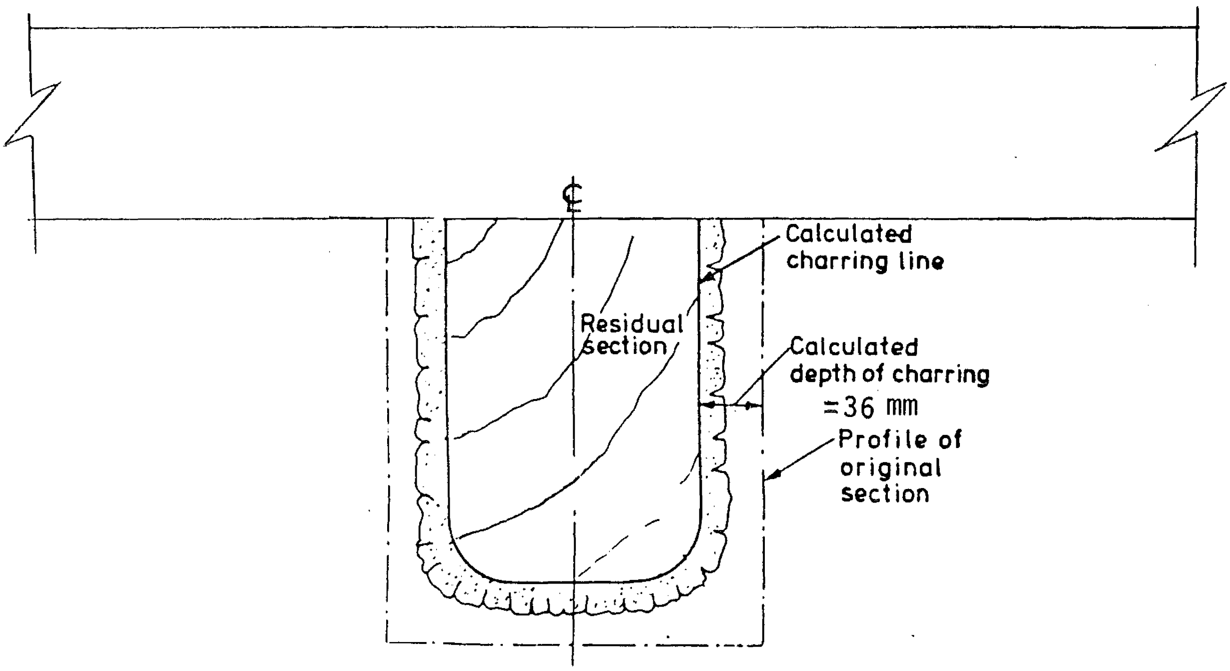


Figure 9.3 Charred section of beam

Actual stress under fire conditions,

$F_b = 50.6 \times 10^6/3.16 \times 10^6 = 16.1$ Mpa

Allowable stress under fire conditions,

$= 2 \times 10.6 = 21.2$ Mpa MP9 (SANZ, 1987)

16.1 Mpa < 21.2 Mpa

Therefore the section can withstand one hour fire assuming that the fire resistant roof provides lateral stability to the fire reduced cross section.

9.6 DESIGN OF KNEE JOINT

The knee joint of the portal is designed using both steel and plywood gussets. Two types of nailing pattern (pattern A and pattern B) are considered in the case of steel gusseted joint. Pattern A is known as 'Gibson type' and pattern B as 'Timbertek type' (TIF, 1989).

9.6.1 STEEL GUSSETED JOINT

STEEL GUSSET

Assume mild steel plate with yield stress $f_y = 250$ Mpa

Permissible stress $F = 250 \times 0.6 = 150$ Mpa

Section modulus required $Z_{req} = M/F = 113.9 \times 10^6/150$

$= .759 \times 10^6$ mm^3

$Z = bd^2/6$ and $d = 585$ mm, Hence $b_{req} = 13.3$ mm

i.e. 7 mm thickness of plate each side.

NAIL DESIGN

Joint moment $M = 113.9$ kN.m

using 3.15 mm diameter, 75 mm long plain shank nails,(as tested)

Basic nail load = 214 N (NZS 3603:1981, table 11)

Allowable load $F = 214 \times \text{factor}$

$= 214 \times 3$ (This factor is taken from 1989 amendment to code)

$= 642$ N

NAILING PATTERN A

After giving edge distances according to figure 10 NZS 3603:1981, the nail layout as shown in figure 9.4 is obtained. Spacing between the rows is 16 mm as indicated in figure.

The distances to the centroid of the nail group are,

$q = 585 - 2 \times 16 - 5 \times 16 \times 2 = 393$ mm

$r = 585 - 38 - 16 - 5 \times 16 \times 2 = 371$ mm

$a = q/r = 1.06$

$G = (1 + a)^3 / (1 + a^2 + 2a \sin \theta)^{0.5}$

and $\theta = 15.524^o$

Hence $G = 5.33$

Pitch $= 2 \times F \times r^2 \times G / 3 \times M$

$= 2 \times 642 \times 371^2 \times 5.33 / 3 \times 113.9 \times 10^6$

$= 2.76$ mm

provide 11 rows of nails @ 30 mm pitch.

Number of nails $= 2\ (393 + 371) / 3 = 509$

NAILING PATTERN B

After giving edge distances according to figure 10 NZS3603:1981, nail layout as shown in figure 9.5 is obtained.

Assuming a spacing of 20 mm between the rows,

The distances to the centroid of the nail group are,

$q = 1280 - 2 \times 20 = 1240$ mm

$r = 545 - 2 \times 20 = 505$ mm

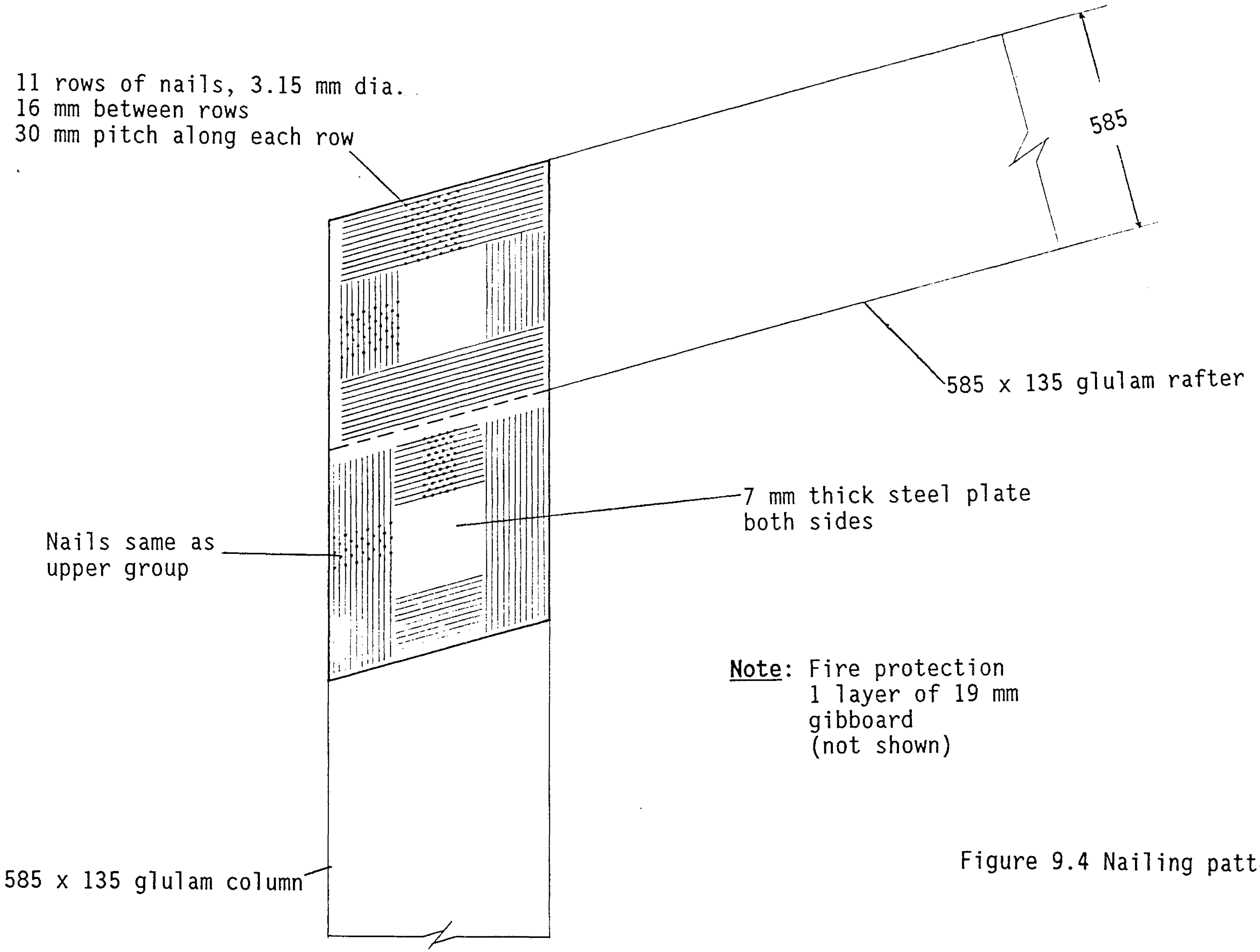
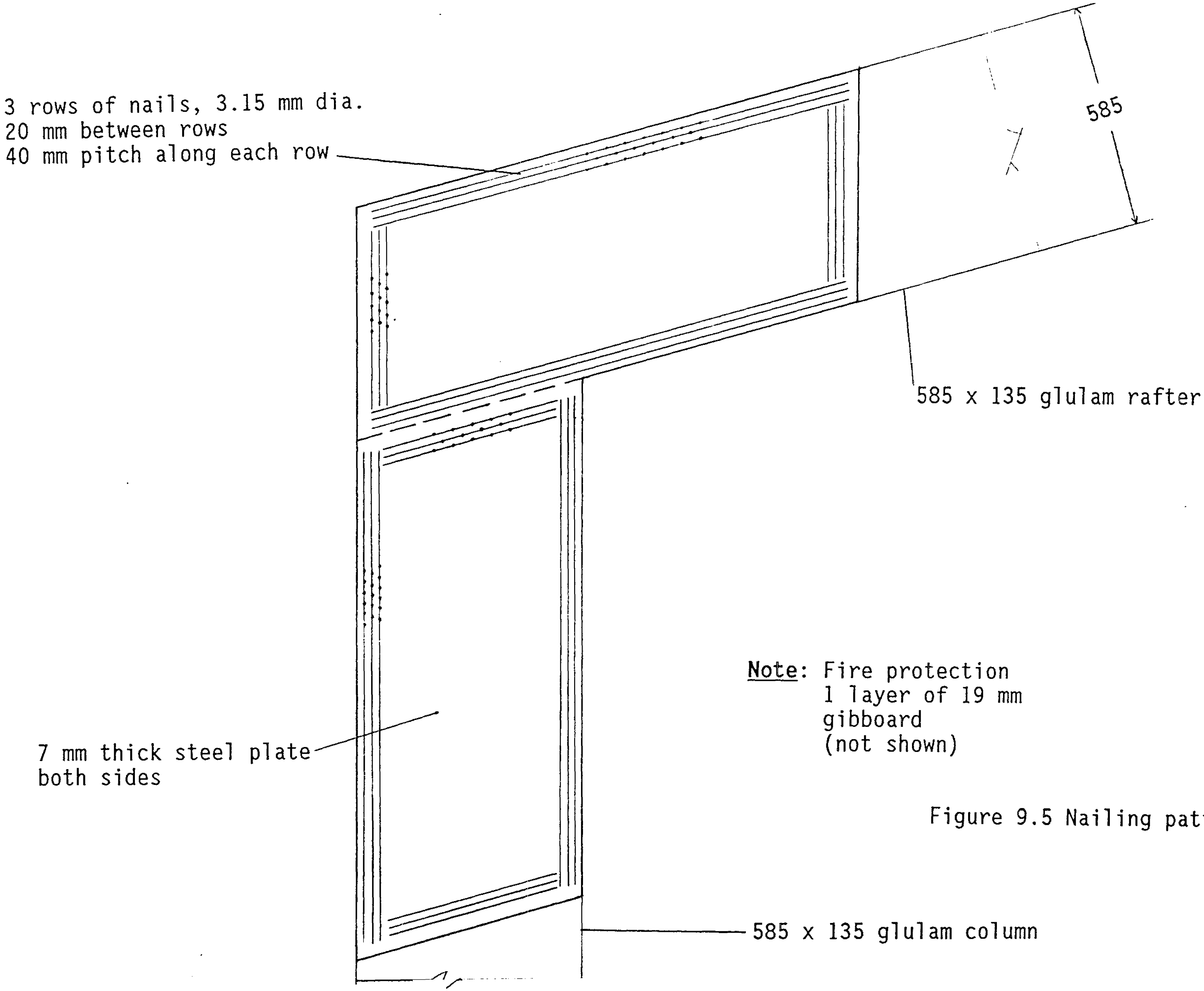


Figure 9.4 Nailing pattern A



Note: Fire protection
1 layer of 19 mm
gibboard
(not shown)

Figure 9.5 Nailing pattern B

$a = q/r = 2.46$

$G = (1 + a)^3 / (1 + a^2 + 2a \sin \theta)^{0.5}$

$\theta = 15.524^o$

Hence $G = 14.32$

Pitch $= 2 \times F \times r^2 \times G / 3 \times M$

$= 2 \times 642 \times 505^2 \times 14.32 / 3 \times 113.9 \times 10^6$

$= 13.72$ mm

provide 3 rows of nails @ 40 mm pitch.

average spacing = 40/3 = 13.3 mm.

Number of nails = 2 (1240 + 505) / 13.72 = 254

9.6.2 THE EFFECT OF JOINT ROTATION ON DISPLACEMENT

The displacement of the knee joint and its rotation for a given displacement of the apex of the portal frame (3 pin frame) are shown.

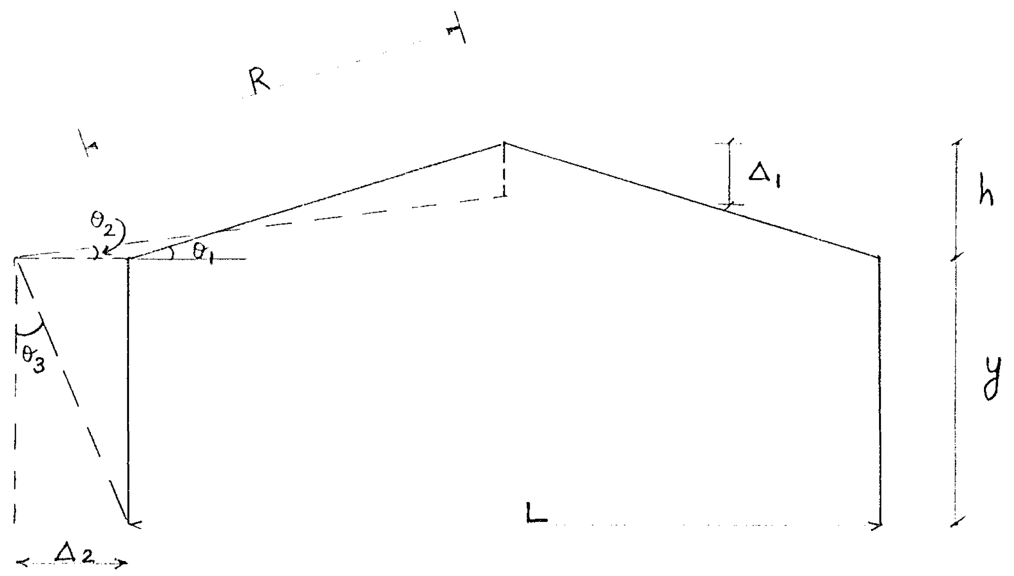


Figure 9.6 Displacements of knee and apex joints

From figure 9.6, the joint rotation

$\delta\theta = 90 + \theta_1 - (90 + \theta_2 - \theta_3) = \theta_1 - \theta_2 + \theta_3$

$= h/0.5L - (h - \Delta_1 / 0.5L + \Delta_2) + \Delta_2 / y$ ....(1)

$\sin \theta_2 = (h - \Delta_1) / R$ ....(2)

$\Delta_2 = (h - \Delta_1 / \tan \theta_2) - L / 2$ ....(3)

For a given displacement of the apex the angle θ_2 can be calculated from equation 2 and Δ_2 can then be calculated from equation 3.

The joint rotation can then be computed from equation 1.

9.6.3 PERFORMANCE OF STEEL GUSSETED JOINT

The joint is analysed using the model described in chapter 8. The nailgroup details together with the expression for load-slip characteristics of nails under cold and fire conditions are input into the computer program to determine the moment-rotation characteristics of the joint. Thus the rotation of the joint corresponding to the design moment can be worked out, from which the deflection of the joint can, then be computed. The details of the procedure are indicated below.

COLD CONDITIONS

(NAILING PATTERN A)

Input data for the computer model, (see figure 9.4)

Breadth of nailing pattern	BB	= 585 - 2 x 16 = 553 mm
Depth of nailing pattern	DD	= 585 - 38 - 16 = 531 mm
Spacing along breadth	SPB	= 30 mm
Spacing along depth	SPD	= 30 mm
Number of rows of nails	NR	= 11
Type of nailing pattern		: staggered

Load-slip expression under cold conditions is,

$P = (0.96 + 0.405w)\ (1 - \exp(-6.06w/0.96))$

where, P = load and w = displacement

The resulting moment-rotation curve is shown in figure 9.7.

<u>NAILING PATTERN B</u>

<u>Input data for the computer model</u>

From figure 9.5,

Breadth of nailing pattern BB = 1240 + 2 x 20 = 1280 mm

Depth of nailing pattern DD = 505 + 2 x 20 = 545 mm

Spacing along breadth SPB = 40 mm

Spacing along depth SPD = 40 mm

Number of rows of nails NR = 3

Type of nailing pattern : staggered

Load-slip expression under cold conditions is,

$P = (0.96 + 0.405w)\ (1 - \exp(-6.06w/0.96))$

where, P = load and w = displacement

The resulting moment-rotation curve is shown in figure 9.8

<u>UNDER FIRE CONDITIONS</u>

Except for the load-slip relationship under fire conditions, the same input data as above is input to determine the performance. The load-slip expression for one hour fire duration is,

$P = (1.119 + 0.016w)\ (1 - \exp(-0.65w/1.119))$

where, P = load and w = displacement

The resulting moment-rotation curves are shown in figures 9.9 and 9.10

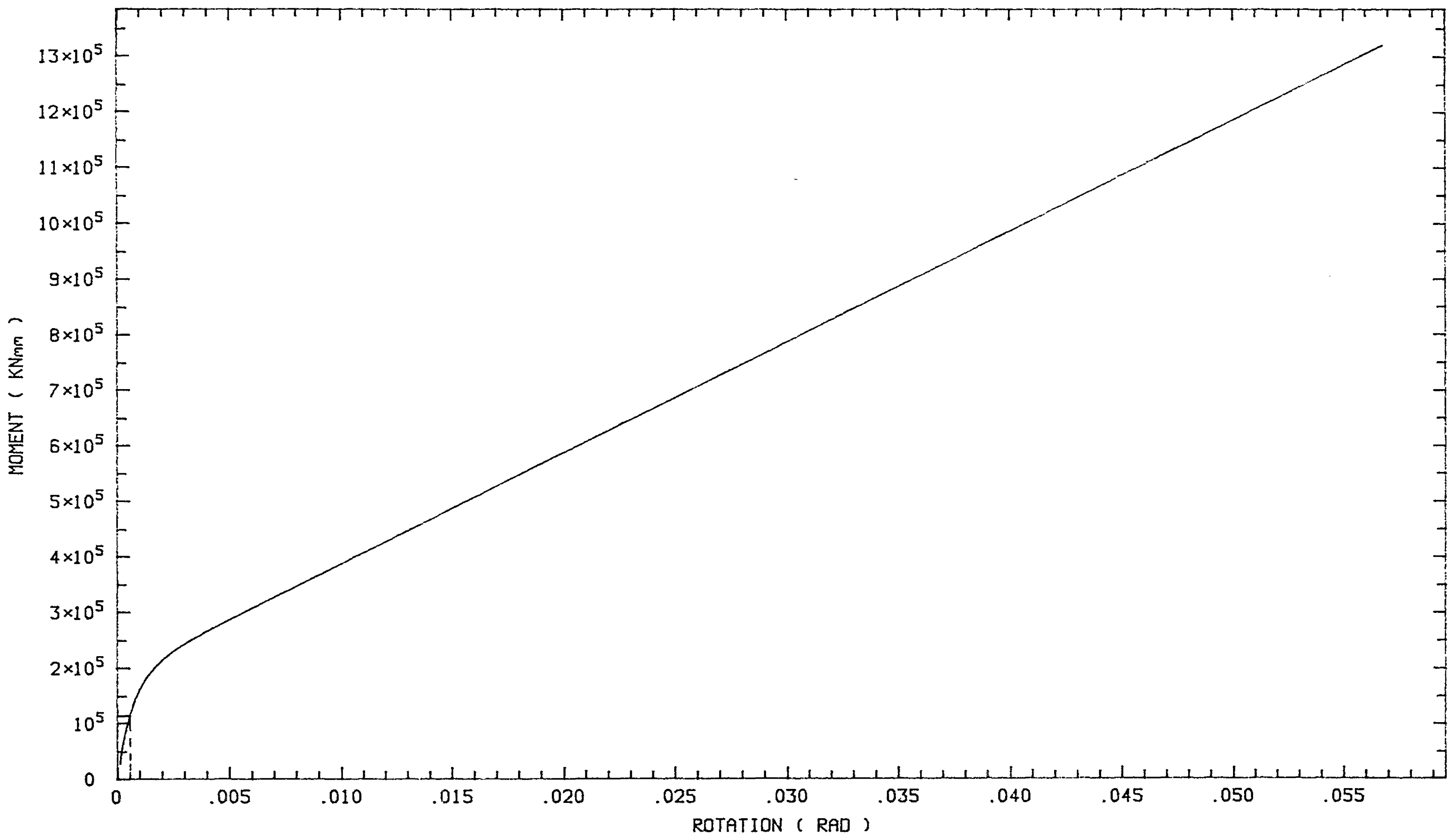


Figure 9.7 Moment-rotation curve for 'Gibson type' nailing pattern

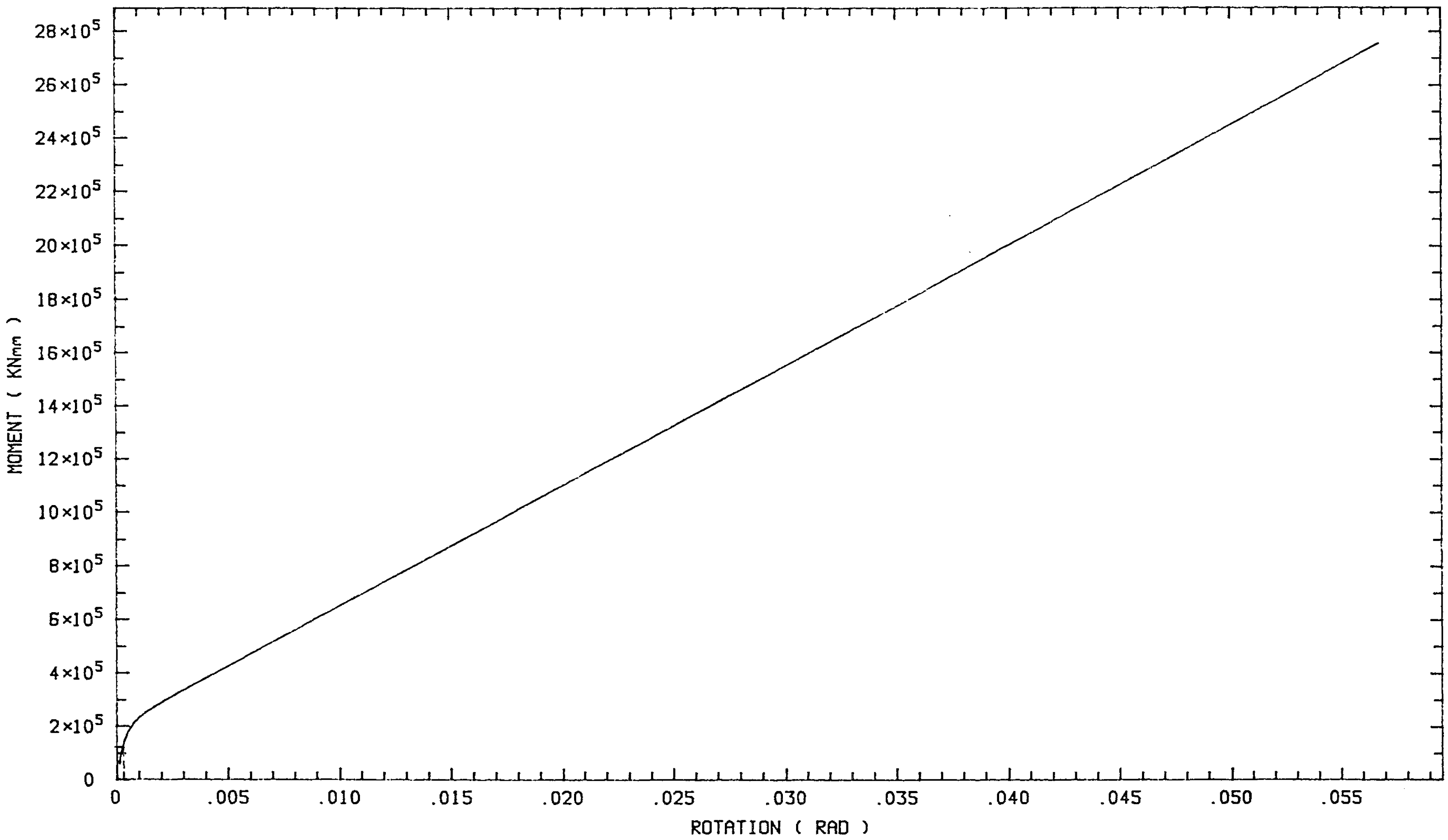


Figure 9.8 Moment-rotation curve for 'Timbertek type' nailing pattern

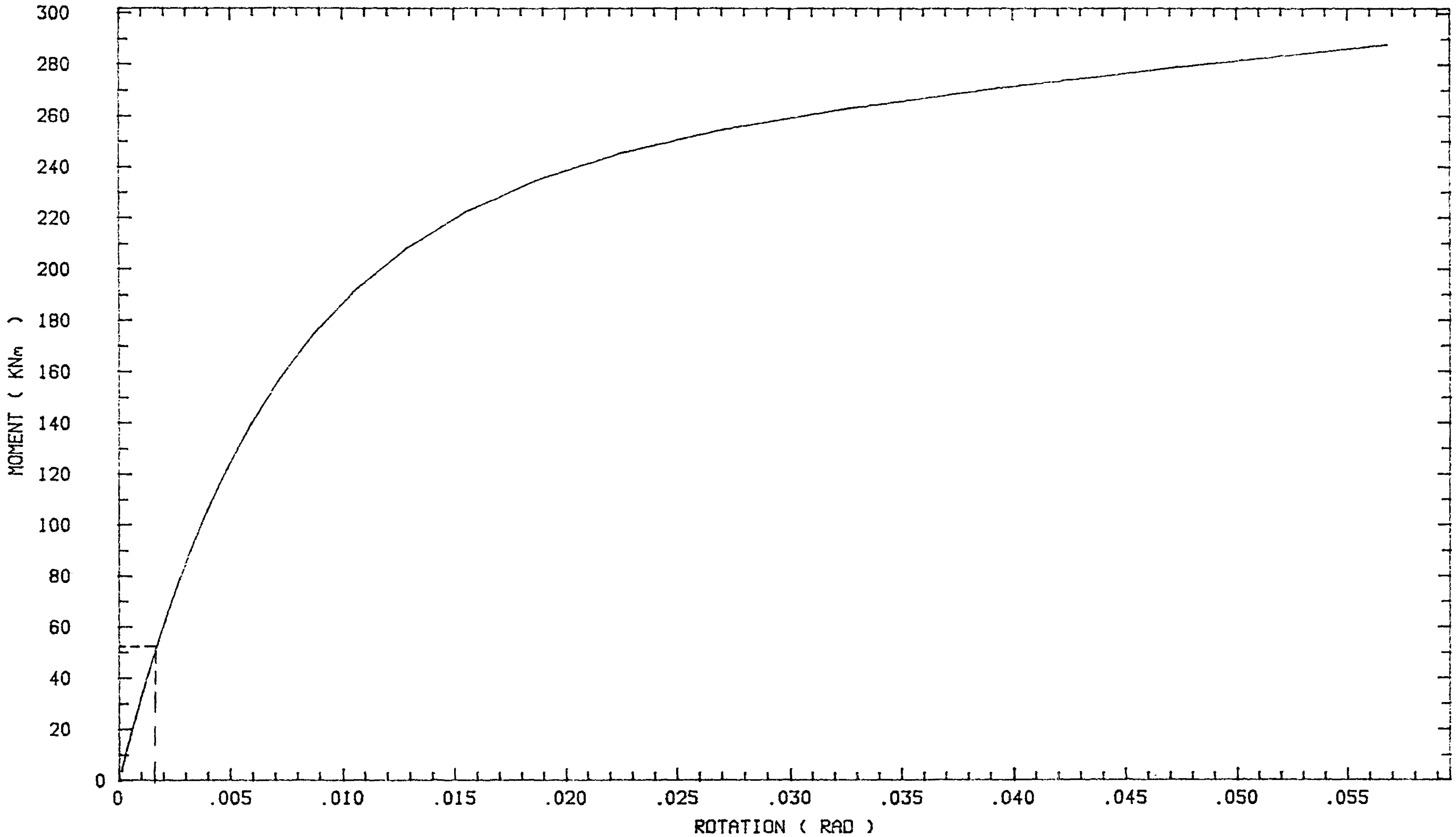


Figure 9.9 Moment-rotation curve for 'Gibson type' nailing pattern under fire conditions

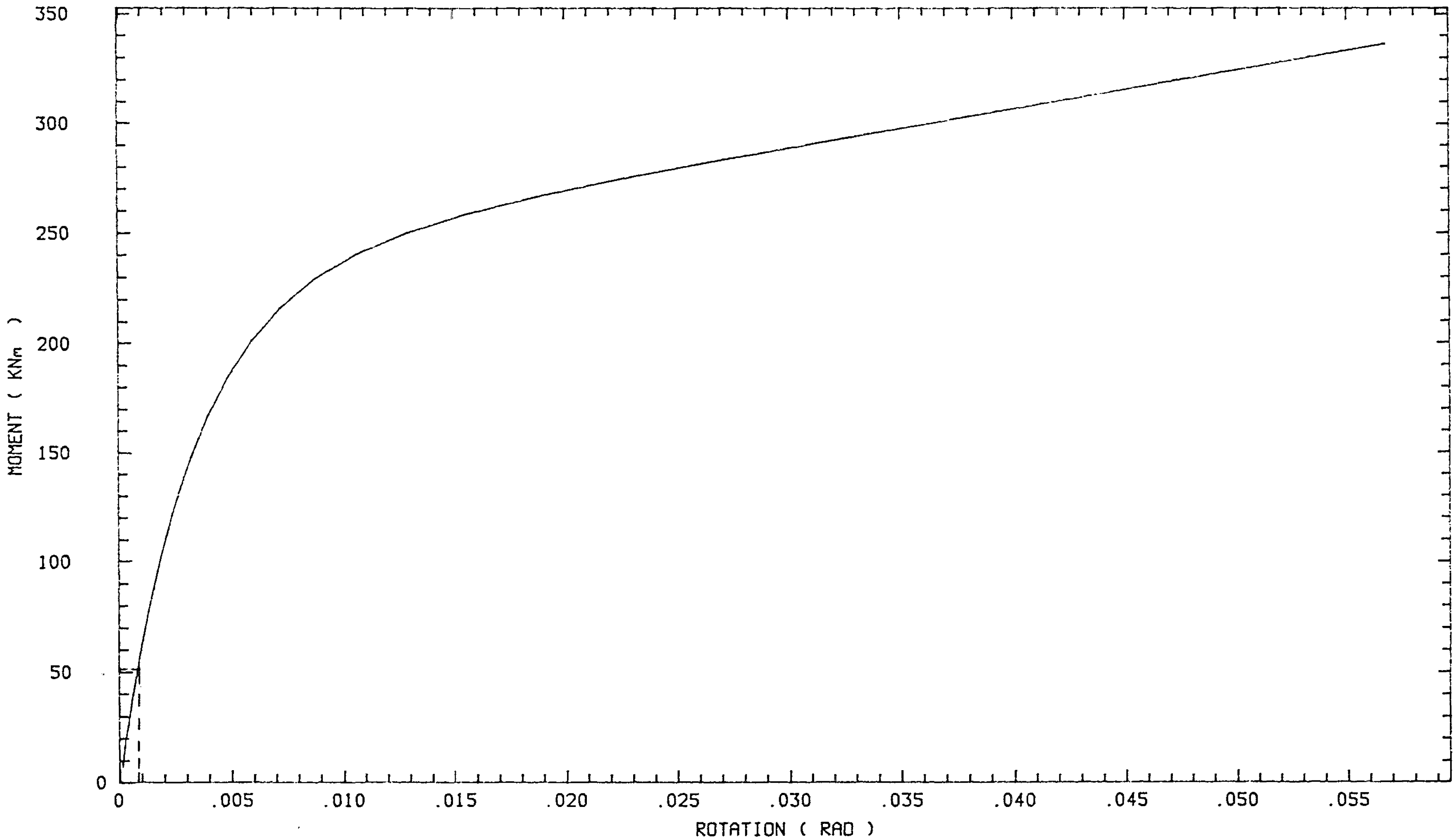


Figure 9.10 Moment-rotation curve for 'Timbertek type' nailing pattern under fire conditions

9.6.4 RESULTS FROM COMPUTER ANALYSIS

The results from computer analysis for the design moment under cold and fire conditions for both the nailing patterns are indicated in table 9.1.

TABLE 9.1 : COMPUTER ANALYSIS OF STEEL GUSSETED JOINT - RESULTS

	MOM.	ROT.	δ_{nail}	F_{nail}	Δ_{knee}	Δ_{apex}
COLD CONDITIONS						
PATTERN A	114	.000479	.18	700	.73	2.64
PATTERN B	114	.000241	.17	665	.42	1.54
FIRE CONDITIONS						
	MOM.	ROT.	δ_{nail}	F_{nail}	Δ_{knee}	Δ_{apex}
PATTERN A	51	.00165	.63	347	2.77	9.00
PATTERN B	51	.000799	.56	528	1.12	4.04

MOM. - Joint moment, kN.m

ROT. - Joint rotation, radians

δ_{nail} - maximum nail slip in furthest nail, mm

F_{nail} - maximum nail force in furthest nail, N

Δ_{knee} - displacement of knee joint, mm

Δ_{apex} - displacement of the apex, mm

9.6.5 PLYWOOD GUSSETED JOINT

PLYWOOD GUSSET

Joint moment = 113.9 KN. m

section depth D = 585 mm

Using the method described in (TIF, 1989) and referring to figure 9.11 assume, $L = 2D = 1170$ mm, say 1200 mm.

Thus $y = (L-D)/(1+(1-D/2L)\tan O) = 508.24$ mm

and $k = (y+D/2)/(y+D) = 0.732$

Find thickness of plywood required from the following expression

$f_1 = (24M/tD^2)(k(1-k)/(4k-1))$

for D-D grade plywood, f_1 in bending = 11.0 Mpa and multiplying this by the duration of load factor $k_1 = 1.4$, the left hand side of the above expression can be known. After substituting the relevant figures in the right hand side, the required effective thickness of plywood is

$t = 52.76$ mm

Effective ply thickness $T = t/0.6 = 87.93$ mm

Hence for one side $T = 87.93/2 = 44$ mm

Use 2 sheets of 22.5 mm plywood glued together on each side.

NAIL DESIGN

Choose pneumatically driven 3.33 x 85 mm nails.

Basic nail load = 240 N (NZS 3603:1981, table 11)

Allowable load F = 240 x factor

= 240 x 2.4 (This factor to be taken from a table in 1988 amendment to code)

= 576 N

After giving edge distances according to figure 10 NZS 3603:1981, the nail layout as shown in figure 9.11 is obtained.

Spacing between the rows is assumed to be 20 mm

Nail group dimensions are,

$q = 1200 - 40 - 20 - 2 \times 2 \times 20 = 1060$ mm

$r = 585 - 2 \times 20 - 2 \times 2 \times 20 = 465$ mm

$a = q/r = 2.28$

$G = (1 + a)^3 / (1 + a^2 + 2a \sin \theta)^{0.5}$

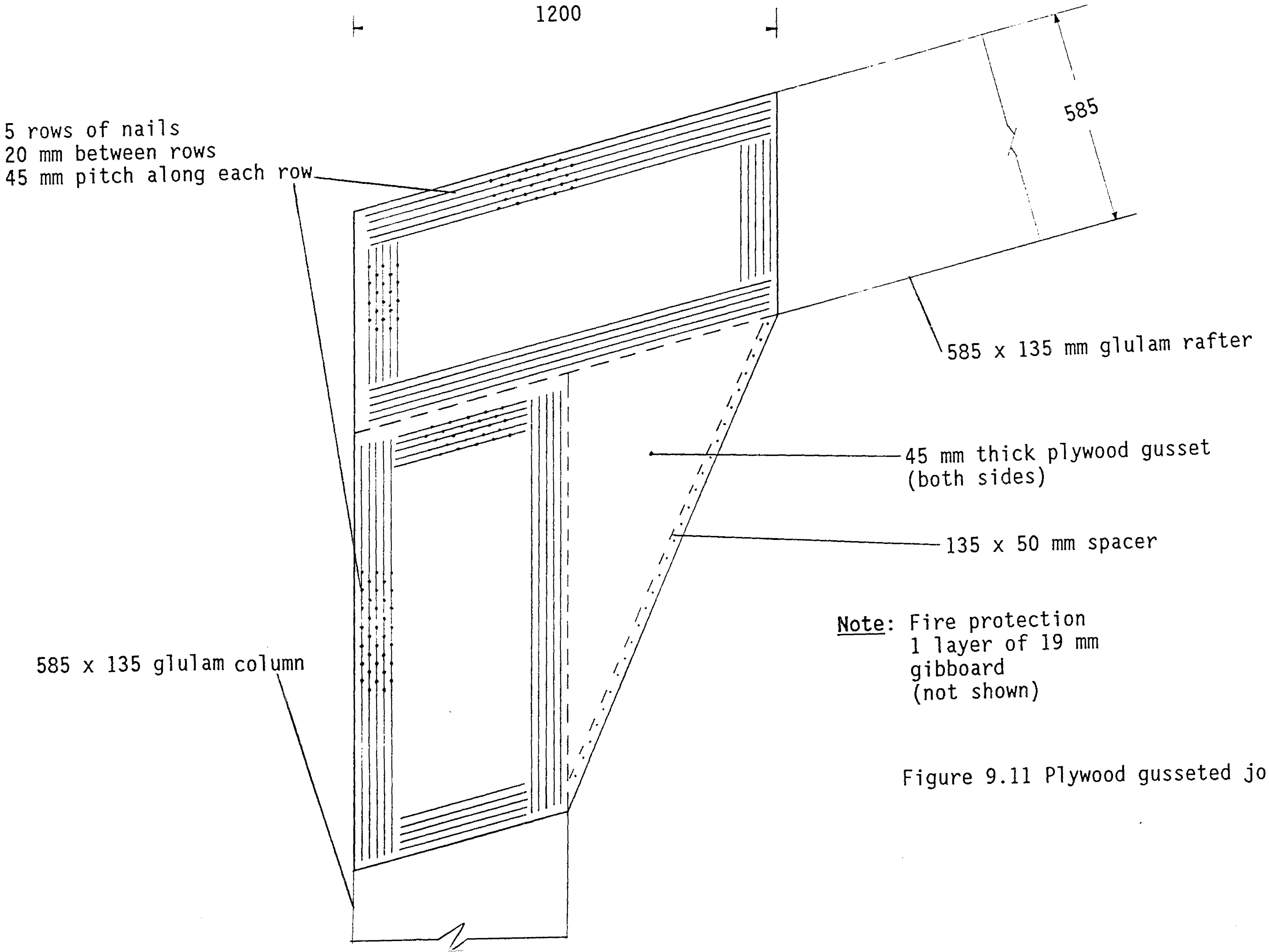


Figure 9.11 Plywood gusseted joint

$\theta = 15.524^{o}$

Hence $G = 12.96$

Pitch $= 2 \times F \times r^2 \times G / 3 \times M$

$= 2 \times 576 \times 465^2 \times 12.96 / 3 \times 113.9 \times 10^6$

$= 9.45$ mm

provide 5 rows of nails @ 45 mm pitch.

Number of nails = 2 (1060 + 465) / 9.45 = 323

9.6.6 PERFORMANCE OF PLYWOOD GUSSETED JOINT

Similar procedure as adopted in steel gusset is used.

COLD CONDITIONS

Input data for the computer model

From figure 9.11

Breadth of nailing pattern BB = 1060 + 4 x 20 = 1140 mm

Depth of nailing pattern DD = 465 + 4 x 20 = 545 mm

Spacing along breadth SPB = 45 mm

Spacing along depth SPD = 45 mm

Number of rows of nails NR = 5

Type of nailing pattern : staggered

Load-slip expression :

$P = (1 + 0.165w)(1 - \exp(-5.15w/1))$

where, P = load and w = displacement

UNDER FIRE CONDITIONS

Except for the load-slip relationship under fire conditions, the same input data as above is used to determine the performance. The load-slip expression for one hour fire duration is,

$$P = (0.28 + 0.06w)\ (1 - \exp\ (-0.3w\ /.28))$$

where, P = load and w = displacement

The resulting moment-rotation curves are shown in figures 9.12 and 9.13.

9.6.7 RESULTS FROM COMPUTER ANALYSIS

The results from the computer analysis for the design moment under cold and fire conditions for both the nailing patterns are indicated in table 9.2. The notation is the same as that used for the steel gusset.

TABLE 9.2 : COMPUTER ANALYSIS OF PLYWOOD GUSSETED JOINT - RESULTS

COLD CONDITIONS					
MOM.	ROT.	δ_{nail}	F_{nail}	Δ_{knee}	Δ_{apex}
114	.000289	0.18	618	0.44	1.6
FIRE CONDITIONS					
MOM.	ROT.	δ_{nail}	F_{nail}	Δ_{knee}	Δ_{apex}
51	.00217	1.36	279	3.32	12.0

9.7 COMPARISON OF PERFORMANCES OF THE GUSSETS

For steel gusset under cold conditions, for a given design moment the joint rotation is doubled under pattern A compared to pattern B. The displacements are also in the same proportion. The nail force in the extreme row of nails is 10% higher for pattern A than for pattern B. This is because the nails are all much closer to the centre of rotation of the joint. Thus very close spacing of rows of nails is not advisable in joint design. The force in the extreme row of nails is about three times the basic nail load. The maximum capacity of these nails as determined from earlier tests is about ten times the basic nail load.

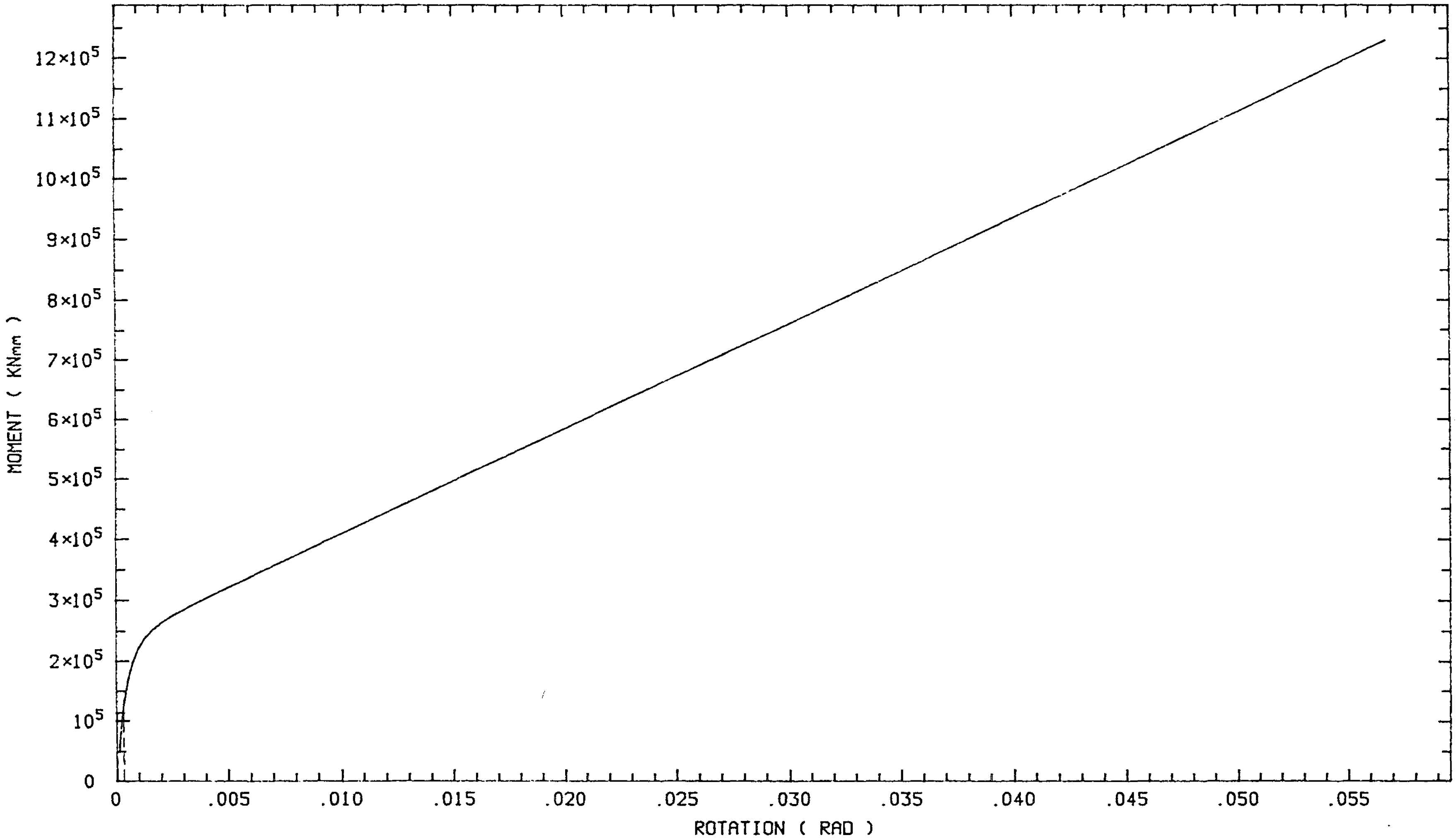


Figure 9.12 Moment-rotation curve for plywood gusseted joint under cold conditions

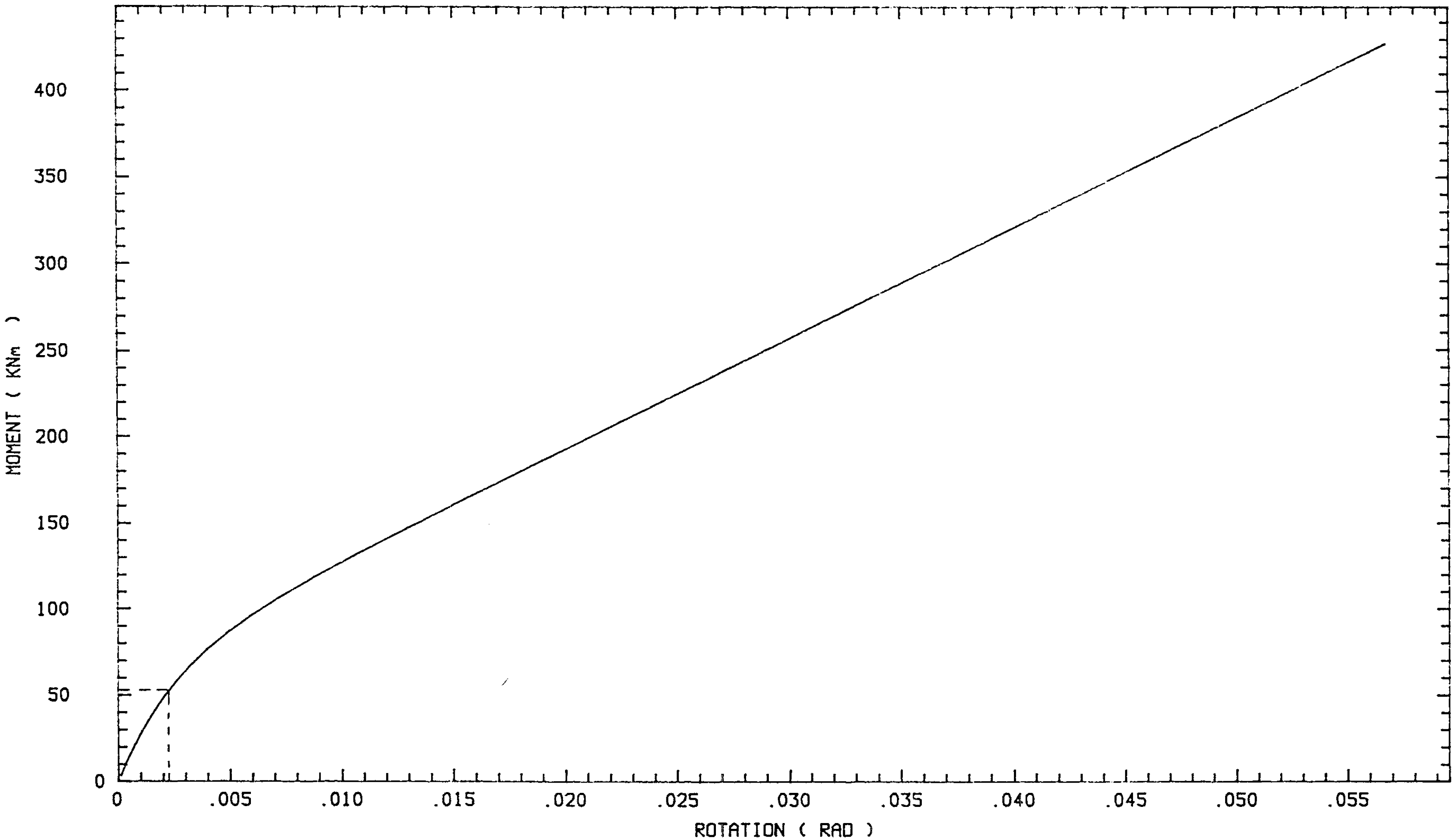


Figure 9.13 Moment-rotation curve for plywood gusseted joint under fire conditions

The behaviour of the joint is quite different when it is exposed to fire. The maximum nail force is about 1.5 times the basic nail load in this case. The failure load of these nails as determined from earlier tests is about six times the basic nail load.

When the joint performance under cold and fire conditions are compared, the joint rotation under fire conditions is observed to be three times that under cold conditions.

In the case of plywood gusseted joint, the rotation under fire conditions is about seven times that under cold conditions. The displacements are also in the same proportion. The maximum nail force under cold conditions is about 2.8 times the basic nail load. The ultimate load carrying capacity of these nails is about ten times the basic nail load.

As the nailing pattern of plywood gusseted joint is similar to nailing pattern B of steel gusseted joint the performances of these two can be compared. Under cold conditions the performances are almost identical. However under fire conditions, the plywood gusseted joint shows more rotations and displacements.

CHAPTER TEN

PREDICTION OF THE FIRE PERFORMANCE OF NAILED TIMBER JOINTS

10.1 INTRODUCTION

An attempt is made to predict the fire performances of nailed on gusseted joints between glulam timber beams, tested under full exposure. This test was carried out at BRANZ. Both steel and plywood gusseted joints were tested. The joints were protected with a single layer of 14.5 mm thick 'FYRESTOP GIBRALTAR BOARD'. This prediction can only be a close approximation due to the type of protection used in our case being different (one layer of 19 mm 'FYRELINE GIBRALTAR BOARD') from that adopted under full exposure.

10.2 TEST DETAILS

The testing arrangement used in the BRANZ tests and the loading details are shown in figures 10.1 and 10.2 respectively.

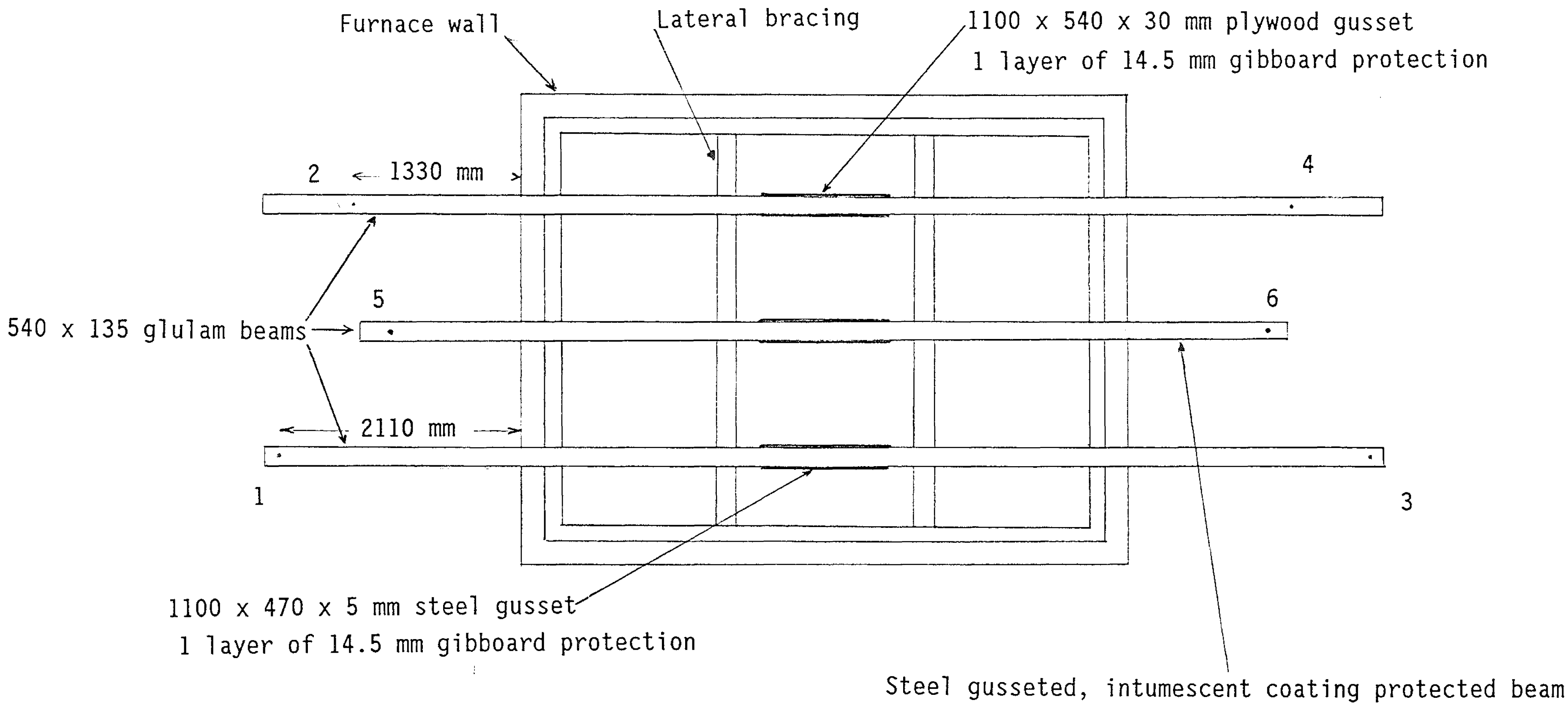
10.3 PERFORMANCE OF STEEL GUSSETED JOINT

The test performance as well as the predicted performance of the gusset is described. The rotation of the joint and the deflection of the beam are computed and compared against that observed during the test.

10.3.1 TEST PERFORMANCE

Plastic rotation of the joint

Figure 10.3 indicates the position of potentiometers (p_2,p_3,p_4 and p_5) placed to determine the displacement of the joint. The displaced position of the joint after the test is shown in figure 10.4.



1,2,3,4,5,6 - Loading points.

Figure 10.1 Test arrangement- plan view.

	load
steel gusseted beam	12.3 kN
plywood gusseted beam	12.5 kN

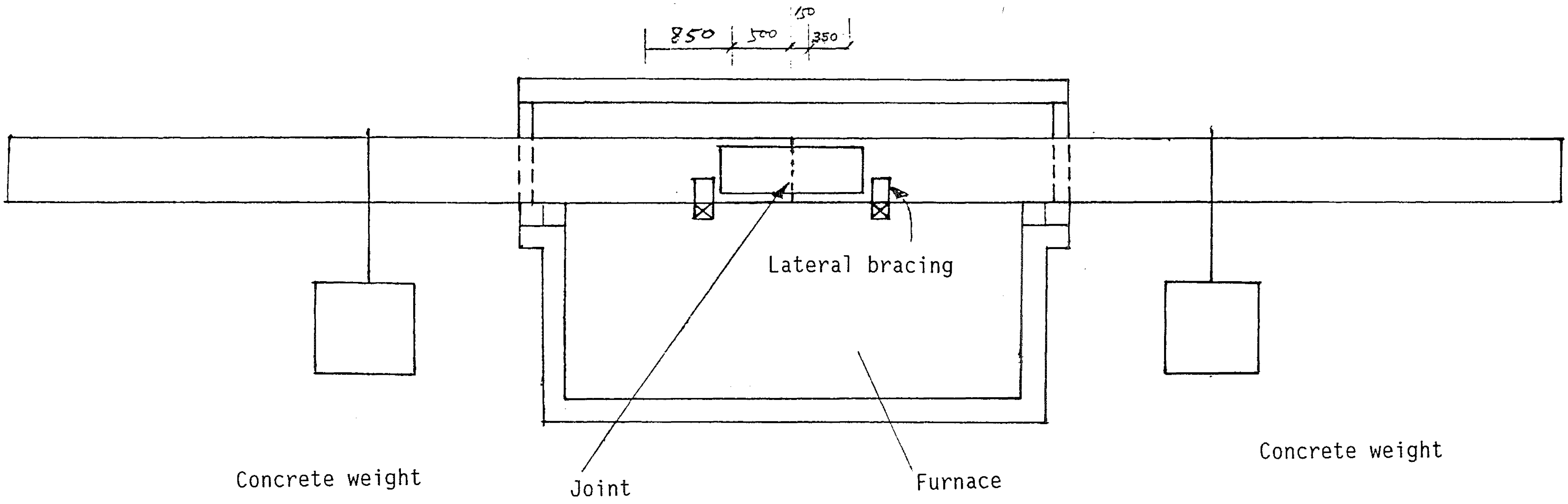


Figure 10.2 Loading details

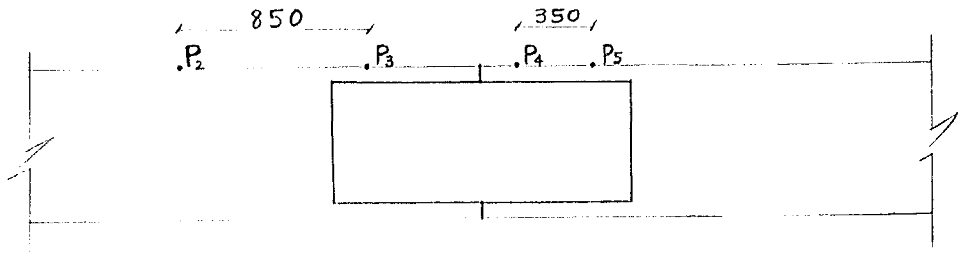


Figure 10.3 Position of potentiometers on the beam

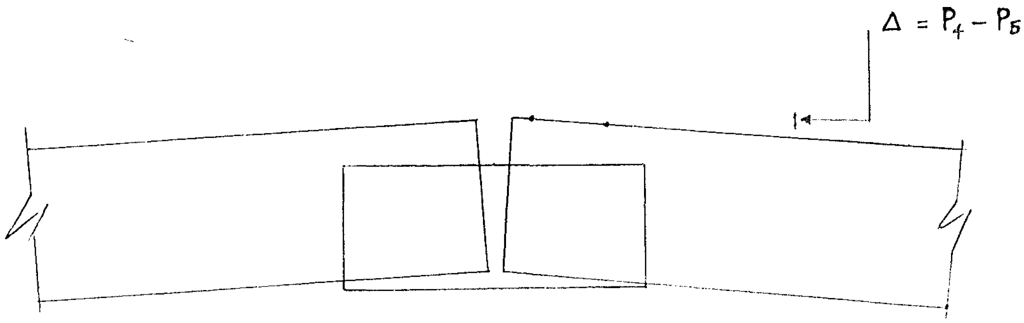


Figure 10.4 Displaced position of the joint

The displacements were recorded during the test. The calculated rotations of the joint from these results against time is indicated in table 10.1 and shown pictorialy in figure 10.5. The reasons for the differences among the three rotations is not clear, but may be due to accuracy in displacement measurements.

10.3.2 PREDICTED PERFORMANCE

Plastic rotation of the joint

When the joint is analysed using the computer model a rotation of 0.0035 radians is indicated for the joint moment of 26 kN.m for 1 hour

Table 10.1: JOINT ROTATION VERSUS TIME FOR STEEL GUSSETED BEAM

Time (min.)	θ_1	θ_2	θ_3
0	0.0038	0	0
5	0.0038	0	0
10	0.0038	0.0012	0
15	0.0043	0.0012	0
20	0.0043	0.0024	0
25	0.0047	0.0024	0
30	0.0052	0.0035	0.0029
35	0.0062	0.0035	0.0029
40	0.0071	0.0047	0.0029
45	0.0076	0.0047	0.0029
50	0.0109	0.0071	0.0057
55	0.0156	0.0141	0.0143
60	0.0209	0.0318	0.0371

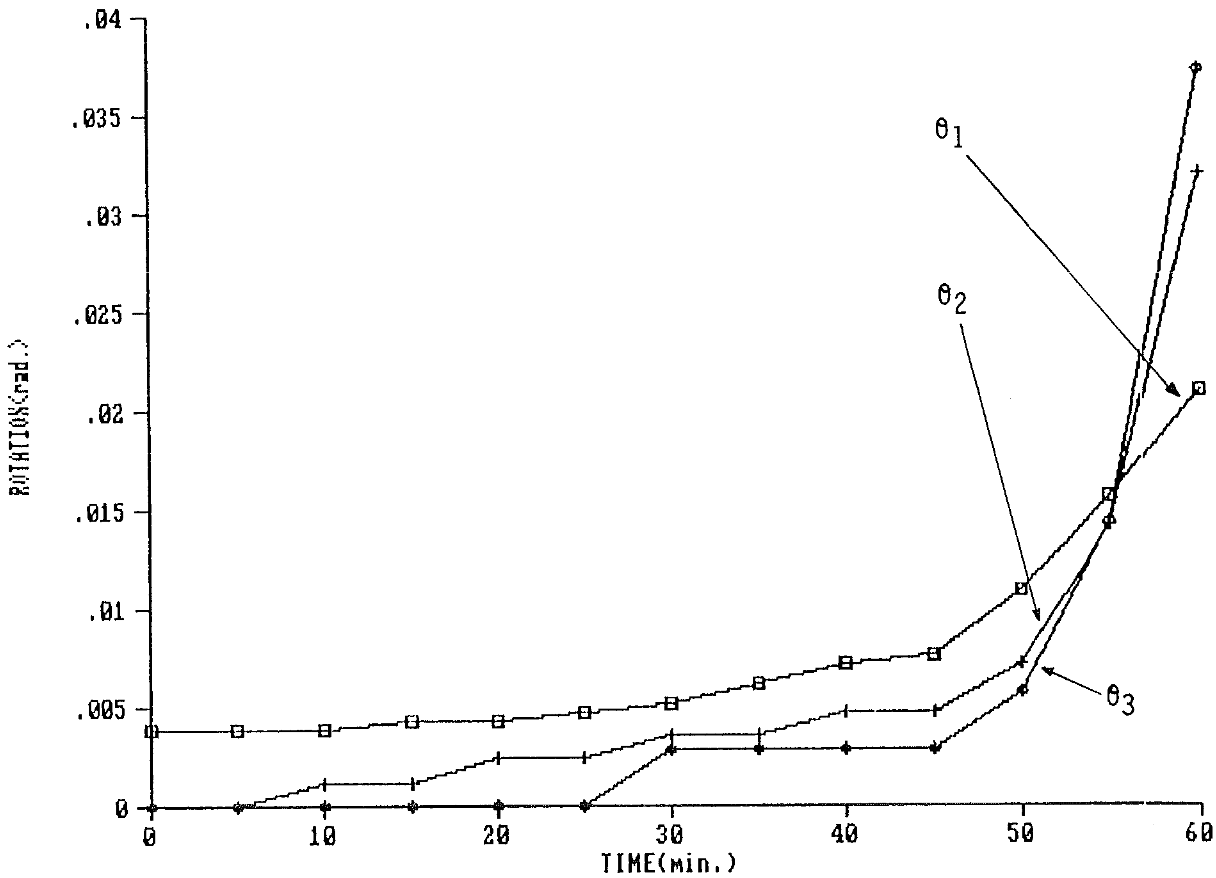


Figure 10.5 Plastic rotation of the joint with time - steel gusseted joint

fire exposure.

10.3.3 COMPARISON

Using 0_3 for comparison, this rotation corresponds to the measured rotation of the joint at 46 minutes. Thus the measured rotation is more than that predicted by the computer model. This could be due to the severity of the fire exposure, and the thinner gibboard protection used in the BRANZ tests.

10.3.4 TEST PERFORMANCE

Deflection of the beam

The total deflection of the loading points of the beam was measured at regular intervals during the test. This deflection at the end of one hour,

For the left end of the loading point = 36 mm and

For the right end = 32 mm

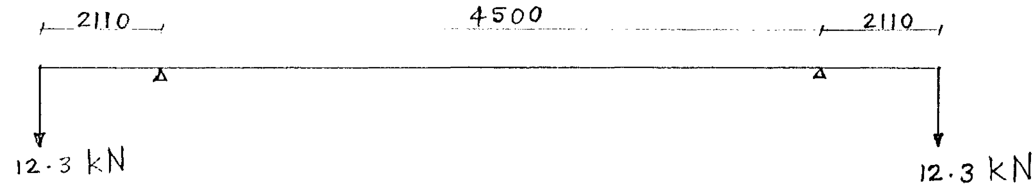
10.3.5 PREDICTED PERFORMANCE

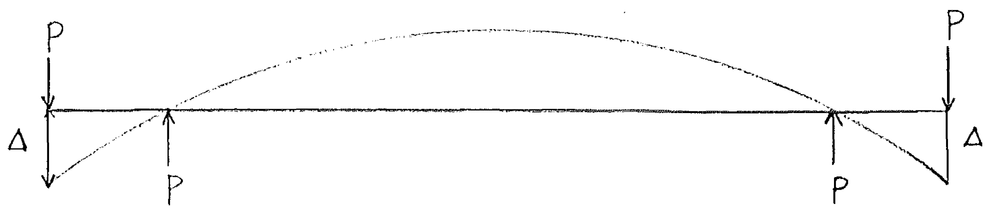
Deflection of the beam

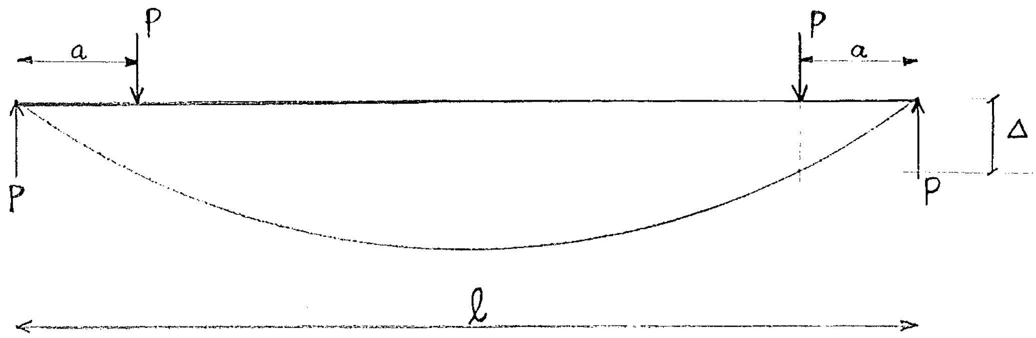
The predicted total deflection of the loading points of the beam consists of predicted elastic deflection of the beam plus the predicted deflection due to plastic rotation of the joint.

Predicted elastic deflection of the beam due to fire conditions

For the given loading the elastic deflection of the beam can be worked out from the section properties of the beam after the fire and the modulus of elasticity of the uncharred portion of the beam. The section properties are worked out assuming a charring rate of 0.6 mm/min. Modulus of elasticity of the uncharred beam is assumed to be 80 % of that under cold conditions. The calculations are shown below.







The deflection of the loading point is given by,

$= Pa^2 (3l - 4a) / 6EI$ where,

$P = 12.3$ kN

$a = 2.11$ m

$l = 8.72$ m

EI_{cold} = 9000 (135 x 540^3)/12 = 1.59 x 10^{13} mm^3

$EI_{after\ fire}$ = 7200 (63 x 468^3)/12 = 3.88 x 10^{12} mm^3

Therefore the corresponding deflections are,

cold = 10.2 mm

after fire = 41.7 mm

<u>Predicted deflection due to plastic rotation of the joint</u>

Deflection of loading point due to joint rotation

= 0.0035 x 2110 = 7.4 mm

<u>Total predicted deflection</u>

Hence total predicted deflection = 41.7 + 7.4 = 49.1 mm

10.3.6 COMPARISON

The measured and the predicted deflections are tabulated below.

	DEFLECTIONS DUE TO		
	JOINT ROTATION	ELASTIC BEAM BEHAVIOUR	TOTAL
PREDICTED	7.4 mm	41.7 mm	49.1 mm
MEASURED			34.0 mm

10.3.7 CONCLUSIONS

It can be concluded from the above results that,

(1) The model predicts a lower deflection for the joint at the end of one hour than that was measured during the test.

(2) The beam deflection resulting from joint rotation appears to be well predicted up to 46 minutes of fire exposure, but beyond that point larger rotations occurred in the test.

(3) The elastic beam deflections appear are significantly less than

those predicted by calculations using a modulus of elasticity of 80% of that under cold conditions.

10.4 PERFORMANCE OF PLYWOOD GUSSETED JOINT

A similar approachas adopted with steel gusset is used.

10.4.1 TEST PERFORMANCE

Plastic rotation of the joint

This is given in table 10.2 and shown pictorialy in figure 10.6.

10.4.2 PREDICTED PERFORMANCE

Plastic rotation of the joint

When the joint is analysed using the computer model a rotation of 0.0041 radians is indicated for the joint moment of 17 kN.m for 1 hour fire exposure.

10.4.3 COMPARISON

Using 0_3 for comparison, this rotation corresponds to the measured rotation of the joint at 31 minutes. Thus the measured rotation is more than that predicted by the computer model. The same reasons as in steel gusset apply here.

10.4.4 COMPARISON OF DEFLECTIONS OF THE BEAM

The measured and predicted deflections are tabulated in the next page. The measured deflection of the left end of the loading point, at the end of one hour is 46 mm and that of the right end at the end of 50 minutes is 17 mm. Thus an average measured total deflection of 31.5 mm is shown in the table below. The workings in the case of predicted deflec-

Table 10.2 : JOINT ROTATION VERSUS TIME FOR PLYWOOD GUSSETED BEAM

Time (min.)	θ_1	θ_2	θ_3
0	0.0045	0	0
5	0.0045	0	0
10	0.0045	0.0012	0
15	0.0045	0.0012	0
20	0.0045	0.0012	0.0012
25	0.0045	0.0024	0.0029
30	0.0045	0.0035	0.0029
35	0.0045	0.0047	0.0057
40	0.0075	0.0059	0.0057
45	0.0090	0.0071	0.0057
50	0.0158	0.0094	0.0057
55	0.0286	0.0110	0.0057
60	0.0391	0.0170	0.0057

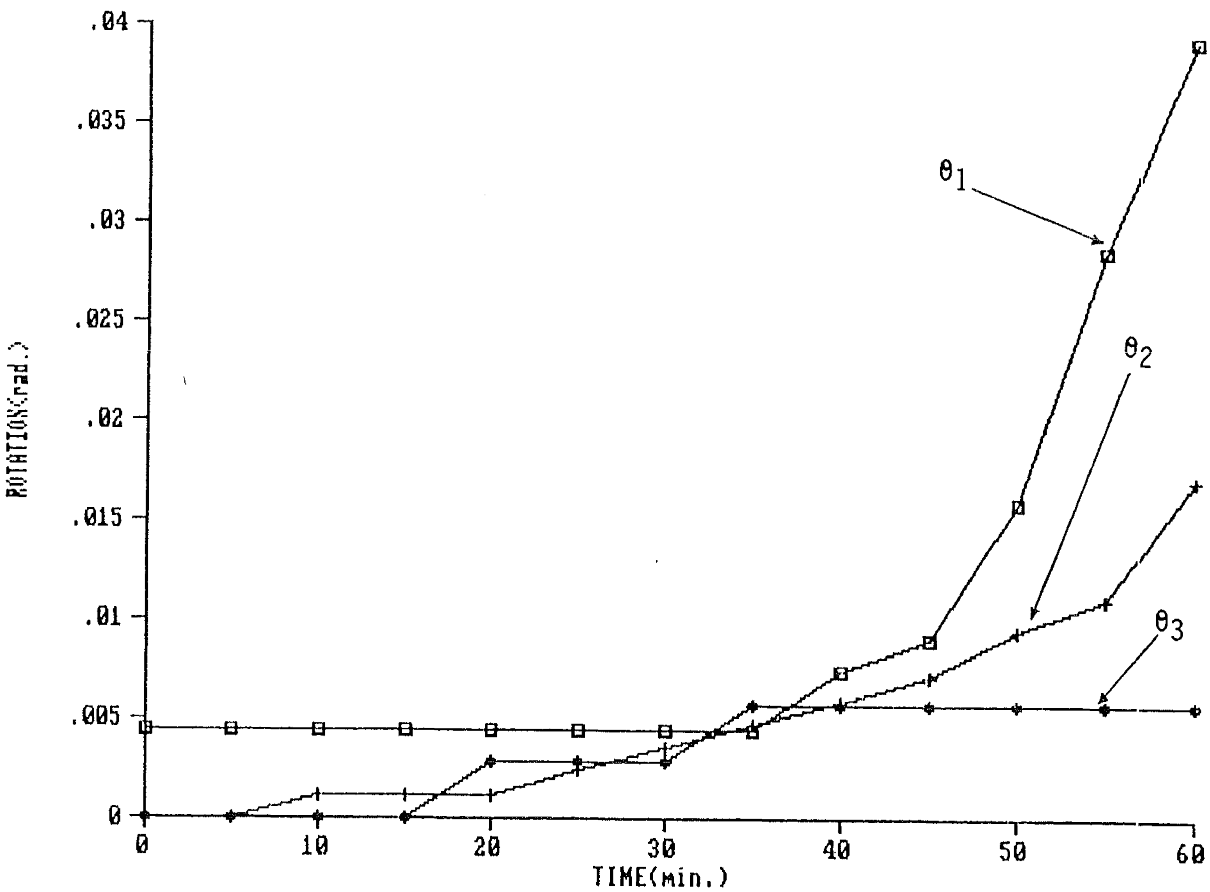


Figure 10.6 Plastic rotation of the joint with time - plywood gusseted joint

tions are similar to the steel gusseted joint.

	DEFLECTIONS DUE TO		TOTAL
	JOINT ROTATION	ELASTIC BEAM BEHAVIOUR	
PREDICTED	5.4 mm	15.4 mm	20.8 mm
MEASURED			31.5 mm

10.4.5 CONCLUSIONS

The following conclusions can be drawn.

(1) The model predicts a lower deflection for the joint at the end of one hour than that was measured during the test.

(2) The beam deflection resulting from joint rotation appears to be well predicted up to 31 minutes of fire exposure, but beyond that point larger rotations occurred in the test.

CHAPTER ELEVEN

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

11.1 SUMMARY

Existing literature in this field (fire performance of nailed joints) has been reviewed.

After successfully simulating the time-temperature results of a BRANZ study, nailed joints were loaded under fire conditions and load-slip curves for nails in both steel and plywood gussets for one hour fire duration were derived. Using this load-deformation behaviour of nails as input information to a computer model, the moment-rotation characteristics of joints were computed. The computer model was used to predict the fire performance of nailed joints tested under full exposure, giving good correlation with test results. A portal frame design example further indicates the application of the results of this study in actual practice.

11.2 CONCLUSIONS

The following conclusions can be drawn from this study.

(1) Nailed gusseted connections between glulam members have good fire performance when protected by gypsum plaster board.

(2) When the joints are protected by two layers of 14.5 mm gypsum plaster board, the temperatures behind the gussets are very low and correspondingly the deflections are also very small. The fire resistance of the joint with this protection is much greater than one hour.

(3) When intumescent coating is used as protection for steel gussets, much higher temperatures are observed behind the gussets causing more charring and more deflection. The fire resistance of this protection is less than one hour.

(4) A relationship between temperature attained on the glulam surface for a given protection and the deflection of the joint when loaded under fire conditions for steel gussets has been established. This would enable a manufacturer to do an unloaded fire test on his protection material and hence know the adequacy of it in a real joint exposed to fire.

(5) Gypsum plaster board having a thickness of 19 mm gave reasonable protection to the joints. For this protection, under maximum expected nail loads (1.8 times basic nail load) the deflection after one hour of fire exposure was 0.75 mm for steel gussets and 1.93 mm for plywood gussets.

After one hour of simulated fire exposure,the nails had an ultimate load capacity of 6 times the basic nail load.

For comparison, under cold conditions the ultimate load capacity of the nails was 10 times the basic nail load.

(5) For similar protection and load levels, plywood gussets produced a deflection more than twice that of steel gussets.

(6) Tests to determine the temperatures along the embedded nail shank indicated the existence of a thermal gradient. For a simulated protection of 19 mm gypsum plaster board the nail temperatures were found to be

too low to cause any serious undesireable effects on joint performance.

Despite high nail head temperatures and adjacent charring of wood surfaces, the temperatures of the embedded ends of the nails were not great, and the connection retained considerable load capacity.

(7) Tests at BRANZ of full scale moment resisting connections gave similar results to those predicted by this project.

(8) Nailed gusseted joints (steel or plywood) in a portal frame (585 x 135 mm glulam) protected by 19 mm thick gypsum plaster board have been shown to have fire resistance greater than one hour with moderate deflections.

11.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The following recommendations for future research should lead to an improved understanding of the fire performance of nailed joints between glulam members.

(1) Most of the conclusions arrived at from this study are based on the results of single tests only. Considering the variability that may exist in a material like timber, it would be useful to do more tests to improve confidence in these results.

(2) Load-deformation characteristics of nails in a joint protected by more suitable protection (eg. 14.5 mm or 12.5 mm gypsum plaster board) can be studied.

(3) The time-temperature results provided by BRANZ study for this project were obtained by exposing only the protected side of the joint to fire and measuring the appropriate temperatures. The fire severity may be greater if the joint undergoes full exposure. This aspect may be further studied.

(4) The variation of embedment strength of timber and bending strength of the nails with temperature could be studied and from this an ultimate strength model which calculates the load capacity of nails under fire conditions could be developed.

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Classn:

THE FIRE PERFORMANCE OF MOMENT-RESISTING NAILED CONNECTIONS BETWEEN HEAVY GLULAM TIMBER MEMBERS

R. CHINNIAH

ABSTRACT: Nailed on, protected steel and plywood gusseted glulam timber joints are loaded under simulated fire conditions and the resulting nail slip at the end of one hour is used to derive load-deformation relationship for nails. This is then input into a computer model to determine the moment-rotation characteristics of joints using these nails under fire conditions.

Department of Civil Engineering, University of Canterbury

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